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EVALUATION OF PHYSICAL AND NUMERICAL HYDRAULIC MODELS, MASONBOR--ETC(U)
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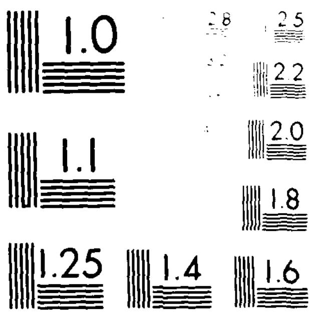
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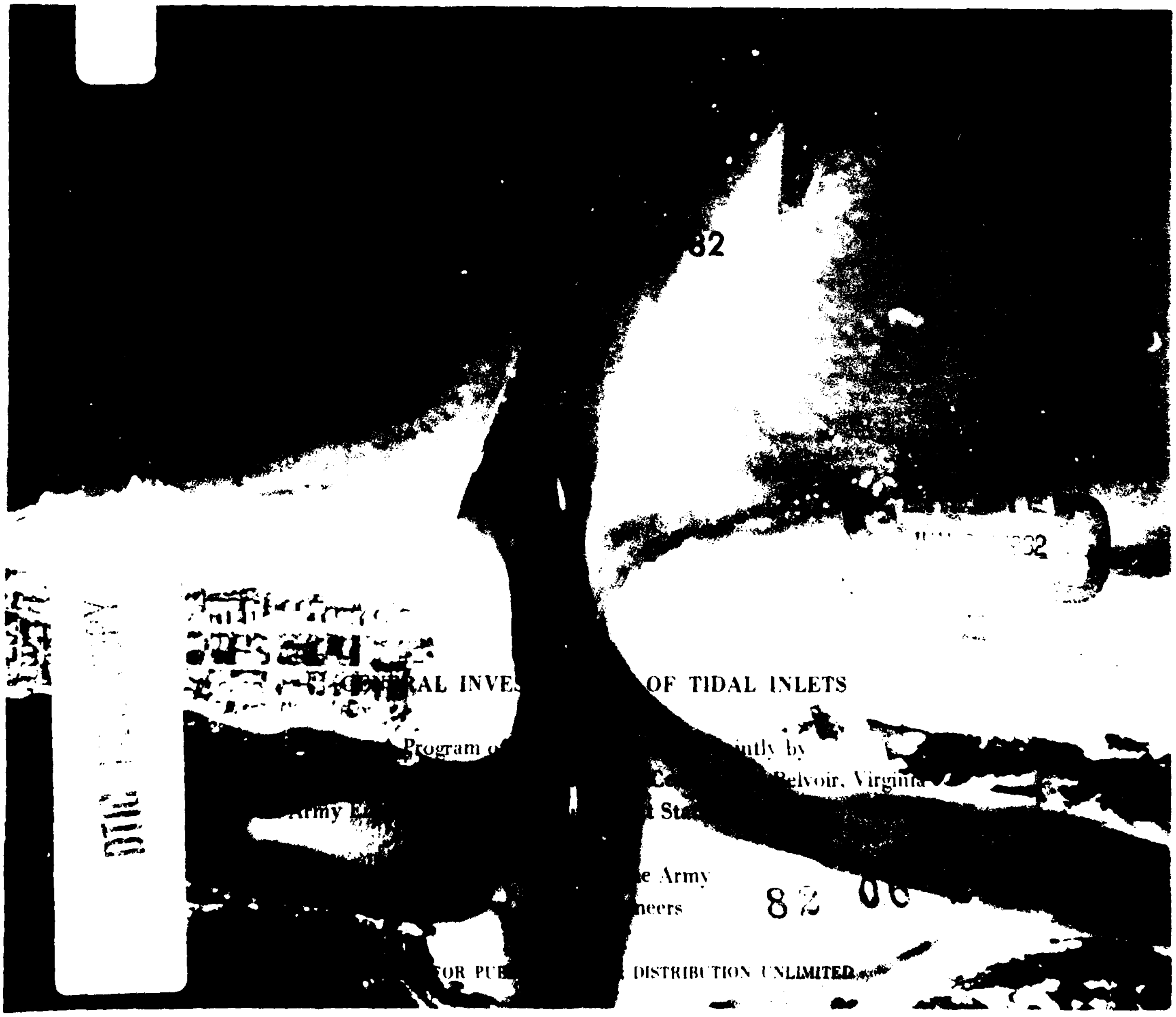
# Evaluation of Physical and Numerical Hydraulic Models, Masonboro Inlet, North Carolina

by

James E. McTamany



Cover Photo: Masonboro Inlet, North Carolina, July 1974

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A fixed-bed distorted-scale physical model, a two-dimensional vertically integrated numerical model, and a spatially integrated numerical model were calibrated to determine prototype conditions at Masonboro Inlet, North Carolina, in September 1969. Comparison of model results with prototype data showed that the physical model and the two-dimensional numerical model reproduced prototype conditions equally well. A second complete set of prototype data, including revised bathymetry in each model, was subsequently obtained at Masonboro Inlet

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in July 1974. After the bathymetry was updated, the models were run using the observed ocean tide as a forcing condition. The model predictions were then compared with prototype data without further recalibration. Both the physical and the two-dimensional numerical models reproduced observed tidal records and vertically averaged velocities equally well. No appreciable improvement in tidal height or velocity predictions was obtained by modeling prototype wind waves in the physical model. The waves caused a slight increase in bay water levels that also occurred in the prototype. Neither numerical model had the capability to model wind waves. The spatially integrated model only predicts the average bay water level and the inlet mean velocity time histories. The predictions from the other models and the prototype data were averaged for comparison with the spatially integrated model. The spatially integrated model did not predict the average bay levels as well as the other models; however, it did predict the mean inlet velocities significantly better than the other two models. The accuracy of the spatially integrated model in predicting mean inlet velocities appears to be less sensitive to calibration than the more detailed physical and numerical models tested in this study.

## FOREWORD

This report, prepared by the Coastal Engineering Research Center (CERC) as one in a series of reports from the Corps of Engineers' General Investigation of Tidal Inlets (GITI), concerns the evaluation of physical and numerical models of a tidal inlet performed as part of the inlet hydraulics study of the GITI. The GITI research program is under the technical surveillance of CERC and is conducted by CERC, the U.S. Army Engineer Waterways Experiment Station (WES), other government agencies, and by private organizations.

The report was prepared by James E. McTamany, Coastal Oceanography Branch, Research Division, under the general supervision of R.M. Sorensen, Chief, Coastal Processes and Structures Branch, and C.L. Vincent, Chief, Coastal Oceanography Branch. Dean M.P. O'Brien, Drs. R.G. Dean, R.L. Weggel, and A.T. Ippen (former member, deceased) of the Coastal Engineering Research Board were involved in the planning or review of this report. Review comments were also obtained from C. Mason, Chief, Field Research Facility, CERC, and W.C. Seabergh, WES.

Comments on this publication are invited.

Approved for publication in accordance with Public Law 166, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963.

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| TILFORD C. CREEL | TED E. BISHOP |
| Colonel, Corps of Engineers | Colonel, Corps of Engineers |
| Commander and Director | Commander and Director |
| Waterways Experiment Station | Coastal Engineering Research Center |

## PREFACE

1. The Corps of Engineers, through its Civil Works Program, has sponsored, over the past 23 years, research into the behavior and characteristics of tidal inlets. The Corps' interest in tidal inlet research stems from its responsibilities for navigation, beach erosion prevention and control, and flood control. Tasked with the creation and maintenance of navigable U.S. waterways, the Corps dredges millions of cubic yards of material each year from tidal inlets that connect the ocean with bays, estuaries, and lagoons. Design and construction of navigation improvements to existing tidal inlets are an important part of the work of many Corps' offices. In some cases, design and construction of new inlets are required. Development of information concerning the hydraulic characteristics of inlets is important not only for navigation and inlet stability, but also because inlets, by allowing for the ingress of storm surges and egress of flood waters, play an important role in the flushing of bays and lagoons.

2. A research program, the General Investigation of Tidal Inlets (GITI), was developed to provide quantitative data for use in design of inlets and inlet improvements. It is designed to meet the following objectives:

> To determine the effects of wave action, tidal flow, and related phenomena on inlet stability and on the hydraulic, geometric, and sedimentary characteristics of tidal inlets; to develop the knowledge necessary to design effective navigation improvements, new inlets, and sand transfer systems at existing tidal inlets; to evaluate the water transfer and flushing capability of tidal inlets; and to define the processes controlling inlet stability.

3. The GITI is divided into three major study areas: (a) inlet classification, (b) inlet hydraulics, and (c) inlet dynamics.

*a. Inlet Classification.* The objectives of the inlet classification study are to classify inlets according to their geometry, hydraulics, and stability, and to determine the relationships that exist among the geometric and dynamic characteristics and the environmental factors that control these characteristics. The classification study keeps the general investigation closely related to real inlets and produces an important inlet data base useful in documenting the characteristics of inlets.

*b. Inlet Hydraulics.* The objectives of the inlet hydraulics study are to define tide-generated flow regime and water level fluctuations in the vicinity of coastal inlets and to develop techniques for predicting these phenomena. The inlet hydraulics study is divided into these areas: (1) idealized inlet model study, (2) evaluation of state-of-the-art physical and numerical models, and (3) prototype inlet hydraulics.

(1) The Idealized Inlet Model. The objectives of this model study are to determine the effect of inlet configurations and structures on discharge, head loss, and velocity distribution for a number of realistic inlet shapes and tide conditions. An initial set of tests in a trapezoidal inlet was conducted

between 1967 and 1970. However, in order that subsequent inlet models are more representative of real inlets, a number of "idealized" models representing various inlet morphological classes are being developed and tested. The effects of jetties and wave action on the hydraulics are included in the study.

(2) Evaluation of State-of-the-Art Modeling Techniques. The objectives of this part of the inlet hydraulics study are to determine the usefulness and reliability of existing physical and numerical modeling techniques in predicting the hydraulic characteristics of inlet-bay systems, and to determine whether simple tests, performed rapidly and economically, are useful in the evaluation of proposed inlet improvements. Masonboro Inlet, North Carolina, was selected as the prototype inlet which would be used along with hydraulic and numerical models in the evaluation of existing techniques. In September 1969 a complete set of hydraulic and bathymetric data was collected at Masonboro Inlet. Construction of the fixed-bed physical model was initiated in 1969, and extensive tests have been performed since then. In addition, three existing numerical models were applied to predict the inlet's hydraulics. Extensive field data were collected at Masonboro Inlet in August 1974 for use in evaluating the capabilities of the physical and numerical models.

(3) Prototype Inlet Hydraulics. Field studies at a number of inlets are providing information on prototype inlet-bay tidal hydraulic relationships and the effects of friction, waves, tides, and inlet morphology on these relationships.

c. *Inlet Dynamics*. The basic objective of the inlet dynamics study is to investigate the interactions of tidal flow, inlet configuration, and wave action at tidal inlets as a guide to improvement of inlet channels and nearby shore protection works. The study is subdivided into four specific areas: (1) model materials evaluation, (2) movable-bed modeling evaluation, (3) reanalysis of a previous inlet model study, and (4) prototype inlet studies.

(1) Model Materials Evaluation. This evaluation was initiated in 1969 to provide data on the response of movable-bed model materials to waves and flow to allow selection of the optimum bed materials for inlet models.

(2) Movable-Bed Model Evaluation. The objectives of this study is to evaluate the state-of-the-art of modeling techniques, in this case movable-bed inlet modeling. Since, in many cases, movable-bed modeling is the only tool available for predicting the response of an inlet to improvements, the capabilities and limitations of these models must be established.

(3) Reanalysis of an Earlier Inlet Model Study. In 1957, a report entitled, "Preliminary Report: Laboratory Study of the Effect of an Uncontrolled Inlet on the Adjacent Beaches," was published by the Beach Erosion Board (now CERC). A reanalysis of the original data is being performed to aid in planning of additional GITI efforts.

(4) Prototype Dynamics. Field and office studies of a number of inlets are providing information on the effects of physical forces and

artificial improvements on inlet morphology. Of particular importance are studies to define the mechanisms of natural sand bypassing at inlets, the response of inlet navigation channels to dredging and natural forces, and the effects of inlets on adjacent beaches.

4. This report discusses the inlet hydraulics of the GITI research program, specifically the evaluation of state-of-the-art modeling techniques. Reports by Harris and Bodine (1977), Masch, Brandes, and Reagan (1977), Sager and Seabergh (1977), and Seelig, Harris, and Herchenroder (1977) in the GITI series discussed the calibration of a physical model and several numerical models of Masonboro Inlet, North Carolina, for the inlet's condition in September 1969. This report presents a comparison of the predictions of the physical model, the two-dimensional numerical model of Masch, Brandes, and Reagan (1977), and the spatially integrated numerical model of Seelig, Harris, and Herchenroder (1977) with a set of prototype data for the condition of Masonboro Inlet in July 1974.

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FIGURES

## CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

| Multiply | by | To obtain |
|---|---|---|
| inches | 25.4 | millimeters |
| | 2.54 | centimeters |
| square inches | 6.452 | square centimeters |
| cubic inches | 16.39 | cubic centimeters |
| feet | 30.48 | centimeters |
| | 0.3048 | meters |
| square feet | 0.0929 | square meters |
| cubic feet | 0.0283 | cubic meters |
| yards | 0.9144 | meters |
| square yards | 0.836 | square meters |
| cubic yards | 0.7646 | cubic meters |
| miles | 1.6093 | kilometers |
| square miles | 259.0 | hectares |
| knots | 1.852 | kilometers per hour |
| acres | 0.4047 | hectares |
| foot-pounds | 1.3558 | newton meters |
| millibars | $1.0197 \times 10^{-3}$ | kilograms per square centimeter |
| ounces | 28.35 | grams |
| pounds | 453.6 | grams |
| | 0.4536 | kilograms |
| ton, long | 1.0160 | metric tons |
| ton, short | 0.9072 | metric tons |
| degrees (angle) | 0.01745 | radians |
| Fahrenheit degrees | 5/9 | Celsius degrees or Kelvins[1] |

[1]To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use formula: $C = (5/9)(F - 32)$.

To obtain Kelvin (K) readings, use formula: $K = (5/9)(F - 32) + 273.15$.

EVALUATION OF PHYSICAL AND NUMERICAL HYDRAULIC MODELS, MASONBORO INLET, NORTH CAROLINA

by
James E. McTamany

# I. INTRODUCTION

## 1. Background and Purpose of Study.

The value of physical models in studying coastal hydraulic problems has long been recognized and now numerical models offer a desirable alternative in certain cases. One of the major study areas in the General Investigation of Tidal Inlets (GITI) research program includes evaluating the effectiveness of state-of-the-art numerical and physical models in predicting the hydraulic characteristics of inlet-bay systems. At the time this study was conceived a fixed-bed physical model of Masonboro Inlet, North Carolina, was being planned at the U.S. Army Engineer Waterways Experiment Station (WES). A request for proposals (bids) was initiated upon the suggestion of the Coastal Engineering Research Board (CERB) to obtain a numerical model of the same inlet.

Masch, Brandes, and Reagan (1977) and Chen and Hembree (1977) developed two numerical (two-dimensional) models which solve the vertically integrated equations of fluid motion and continuity. These models predict vertically averaged velocities and water elevations. A lumped-parameter model, developed by Huval and Wintergerst (1977), predicts the inlet mean velocity and average bay tidal elevation time histories. Harris and Bodine (1977) compared the predictive capability of these numerical models with that of the physical model for the condition of the inlet in September 1969. A spatially integrated numerical model, developed by Seelig, Harris, and Herchenroder (1977), predicts mean inlet velocities and average bay levels. This report discusses the predictive capability of the spatially integrated model and one of the two-dimensional numerical models as compared to prototype and physical model data. It should be noted that other numerical models have been applied to Masonboro Inlet (e.g., Amein, 1975 and Butler, 1978); however, the present study was limited to an investigation of models applied to Masonboro Inlet as part of the GITI program.

## 2. Study Area.

Features of the inlet-bay system at Masonboro Inlet, North Carolina (Fig. 1), as it existed during 1969 include a weir jetty on the north shore at Wrightsville Beach, a large shoal on the south side of the main channel opposite the jetty, interior mudflats that are extensively flooded during a tidal cycle, and channels leading several miles along the coast on the bay side of the inlet which connect to other inlets north and south of Masonboro Inlet. The weir crest extends to mean sea level (MSL) allowing sand transport and flow over the crest during half of the tidal cycle. Sager and Seabergh (1977a) discuss the history of Masonboro Inlet and provide photos of the inlet from 1945 to 1966.

## 3. Prototype Data Collection.

Prototype data sets, collected in September 1969 and July 1974, consisted of bathymetric surveys, water surface elevations and velocities at selected

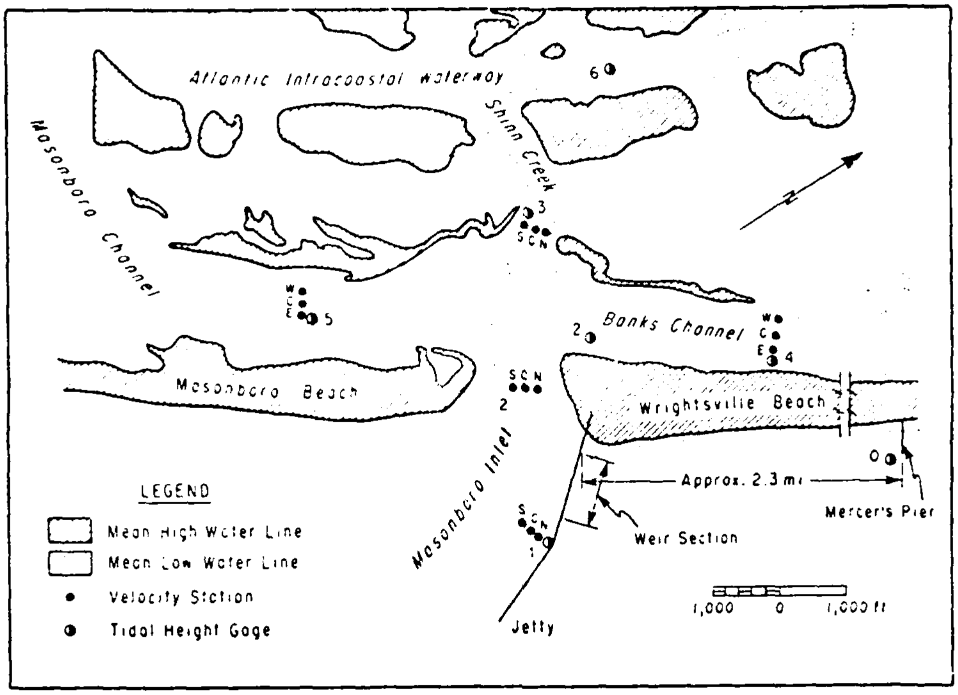


Figure 1. Prototype gage locations at Masonboro Inlet, North Carolina, 1969.

cross sections, and temperature and salinity measurements. General wind and ocean wave conditions were also observed.

Figure 1 shows the locations of the five ranges, three stations within each range, where velocity measurements were made in 1969; stations are labeled S, C, and N for south, center, and north, and W, C, and E for west, center, and east. At each of these stations velocities were recorded at the surface, middepth (when practicable), and bottom. Thus, a maximum of nine velocity readings were taken at any range at any one time. Readings were not taken simultaneously since only one boat and one velocity gage were used per range. The locations of the seven tide gages are also shown in Figure 1.

During the 1974 prototype data collection effort, two additional velocity ranges, labeled 4I and 5I were included in the survey (Fig. 2). A tidal height gage was added at range 5I but no gage was installed in the Intracoastal Waterway as in 1969. The ocean gage was also moved to Crystal Pier, 2 miles (3.2 kilometers) nearer the inlet. The ocean tide level during the 1974 study period averaged 0.3 foot (9 centimeters) lower with a range of 0.25 foot (8 centimeters) greater than the tide observed for the 1969 study. A comparison of the ocean tide records used in the 1969 and 1974 model studies is shown in Figure 3. The major bathymetric changes which occurred between the two conditions were (a) reorientation of the channel and ebb tidal delta just seaward of the inlet throat, (b) seaward growth of the ebb tidal delta, and (c) deepening of some interior channels due to dredging. Bathymetric changes which occurred near the inlet are shown in Figure 4.

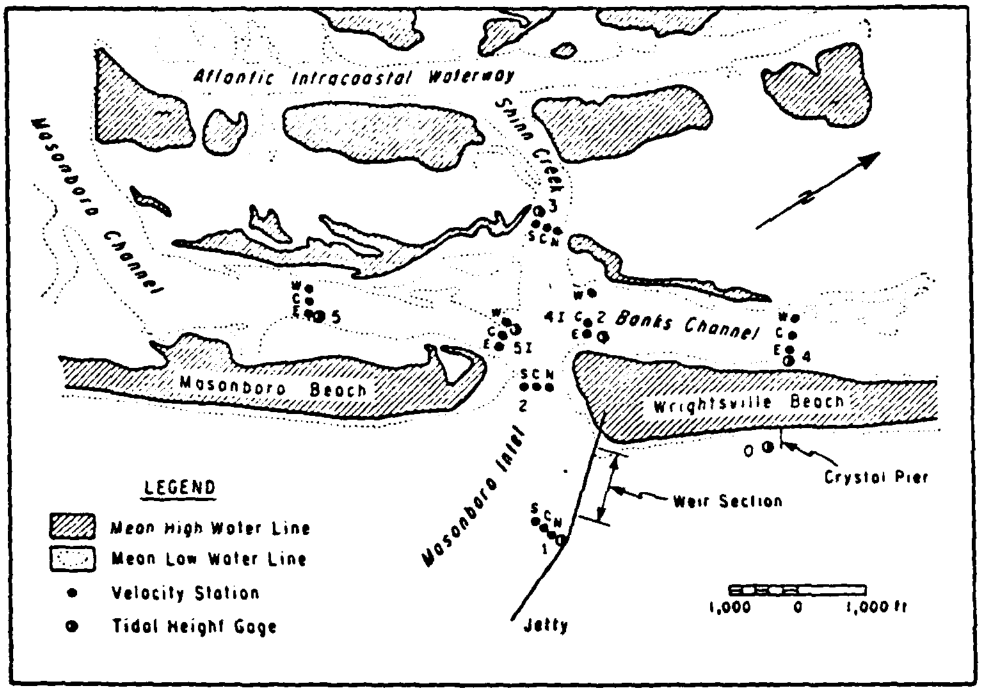


Figure 2. Prototype gage locations at Masonboro Inlet, North Carolina, 1974.

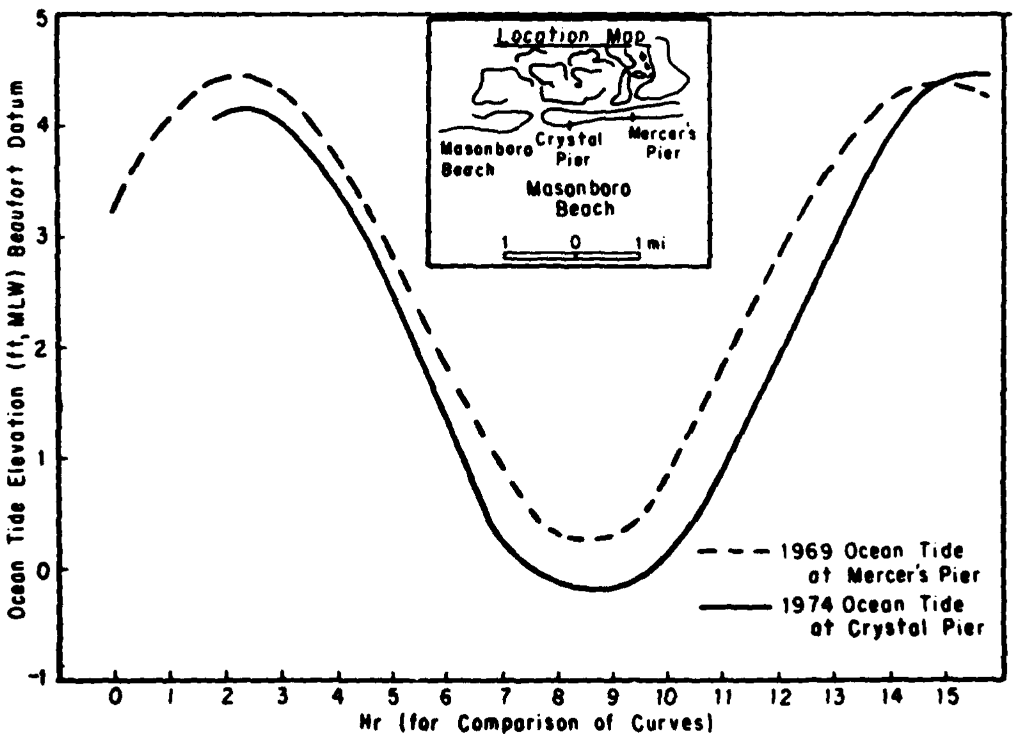


Figure 3. Comparison of prototype ocean tides used in the 1969 and 1974 model studies.

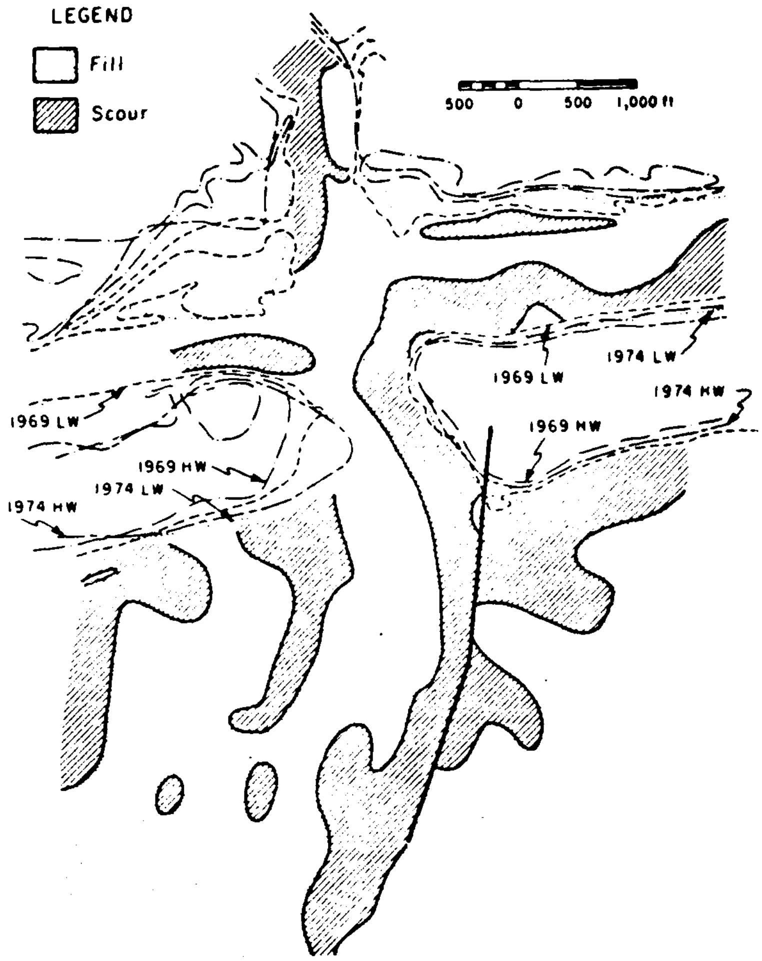


Figure 4. Bathymetric changes at Masonboro Inlet, showing the 1969 and 1974 high water (HW) and low water (LW) levels.

4. Model Development.

In evaluating the models it is necessary to distinguish between the two procedural steps used in developing a model for engineering use--calibration and verification. Harris and Bodine (1977) give the following definitions of these terms:

(a) *Calibration*--The process where a hydraulic model is checked with prototype data and systematically adjusted to reproduce the water level and current data from corresponding prototype driving forces.

(b) *Verification*--The process where independent prototype data (data not used in calibration) are used to verify that a calibrated hydraulic model produces satisfactory results.

Calibration is needed to compensate for approximations made in constructing the model and to obtain the proper amount of frictional dissipation by

adjusting model roughness. Verification is needed to independently check that the adjustments made were physically reasonable and not purely artificial. It might be possible to adjust the model in a variety of ways in order to force the model results to fit the prototype calibration data. The greater the amount of prototype data made available to the modeler, the greater the chance he has of choosing the calibration adjustments which best simulate the prototype. In this study, the 1969 survey data were used to calibrate the models and the 1974 data were used to verify them.

Since one of the two-dimensional numerical models was not acceptably calibrated (Chen and Hembree, 1977), it was not run for the 1974 verification condition. The other two-dimensional model (Marsh, Brandes, and Reagan, 1977) is the only two-dimensional model discussed further in this report. The lumped-parameter model was acceptably calibrated, but because of discrepancies between the equations actually solved and those obtained by rigorous development (Harris and Bodine, 1977), it too was not run for the 1974 verification condition. The spatially integrated model of Seelig, Harris, and Herchenroder (1977) was developed after Harris and Bodine's (1977) comparison of model calibrations. Because of its relative ease of use, the model's calibration and verification are also discussed in this report.

5. Summary of Results.

This study demonstrates that accuracies comparable to that of a distorted-scale physical model can be obtained by a two-dimensional, vertically integrated numerical model in predicting water levels and vertically averaged velocities for a typical tidal inlet. The spatially integrated model (Seelig, Harris, and Herchenroder, 1977) is also seen to be an effective tool for predicting inlet mean velocities. The spatially integrated model is easy to apply and considerably cheaper to operate than physical or two-dimensional numerical models, but generally cannot provide as much detail or accuracy as those models.

## II. PHYSICAL MODEL

1. Construction and Operation.

The fixed-bed physical model was constructed at WES to scales of 1:300 horizontally and 1:60 vertically (Fig. 5). Froude number similarity governed the scaling of dynamic variables. In order to include the entire tidal prism in the allotted shelter space, an artificial bending of bay areas north and south of the inlet and a schematization of distant areas of the bay were necessary. Because the model was distorted (i.e., horizontal and vertical scales were not identical), several types of materials were used to provide proper flow resistance to roughness. Raked concrete, stucco, and metal strips were used to generate turbulent frictional losses and mixing; the metal strips could be flattened or raised during calibration to vary model roughness. Water levels were measured using tidal stage transmitters and drum recorders. Velocities were measured using miniature cup meters. A tide generator was used to impose water levels along the model ocean boundary. Two 20-foot-wide (6 meter) monochromatic wave generators were used to simulate observed wave conditions. Since negligible variation in density of the prototype water was observed, freshwater was used in the model. Additional details on the model construction and operation are provided in Sager and Seabergh (1977b).

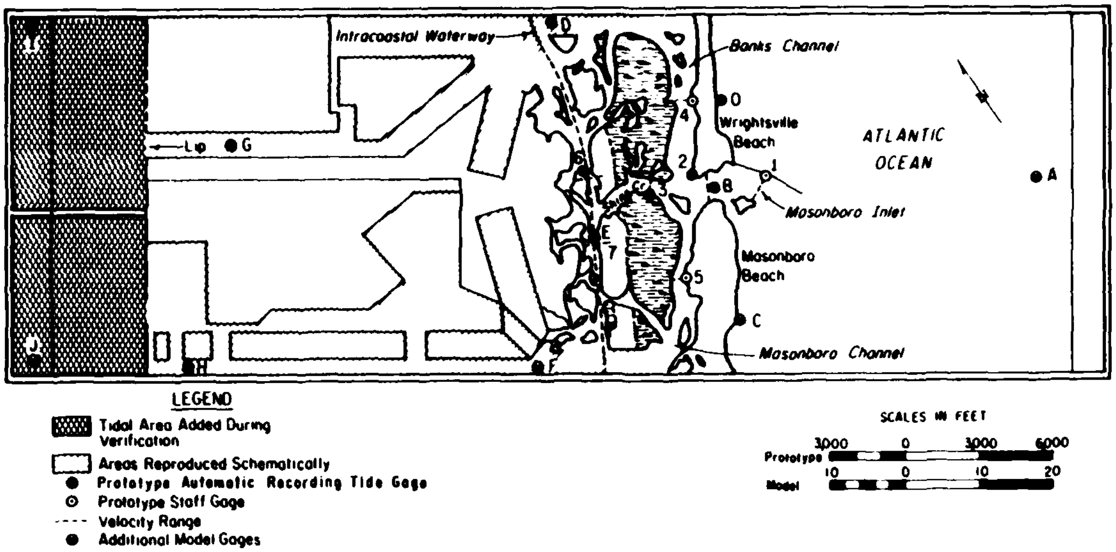


Figure 5. Physical model layout of Masonboro Inlet, North Carolina (Seabergh and Mason, 1975).

2. Calibration.

Since the initial calibration of the model resulted in lower than observed velocities, several adjustments were made to the model. Tidal flows were subsequently increased by enlarging the bay area. Additional control of velocities was achieved by adding and adjusting roughness elements. A decrease in high water elevations was corrected by increasing the ocean high tide 0.2 foot (6 centimeters) prototype scale; a lip was inserted in the bay to restrict flow (see Fig. 5). The model was then considered to be sufficiently calibrated. Typical examples of the calibration achieved for water levels and velocities at tide gage 2 and velocity range 2C are shown in Figures 6 and 7, respectively.

Additional tests were made in the physical model using waves. Since detailed measurements of prototype wave conditions were not made, these tests did not contribute to the calibration of the model. However, these tests did indicate that waves have an important effect on the tidal flow, and that future field and model studies should take waves, if present, into account. Waves 3 feet (0.9 meter) high with a 7-second period prototype scale were used. The wave directions chosen were representative of those most frequently occurring in summer and winter. The introduction of waves resulted in a general increase in bay tide elevations. Properly scaled waves in a distorted-scale physical model can simulate the interaction between the waves and the mean current through radiation stress (Harris and Bodine, 1977). Refraction patterns can be reproduced; however, diffraction scale effects will exist and need to be analyzed to determine if they are significant. Scale distortion effects in reflection and bottom friction must also be analyzed and compensated for if either is shown to be significant. Complete similitude of the surface wave field cannot be achieved in a distorted-scale physical model nor in a undistorted-scale physical or numerical model. Scale effects should be analyzed and the resulting modification of the wave field estimated.

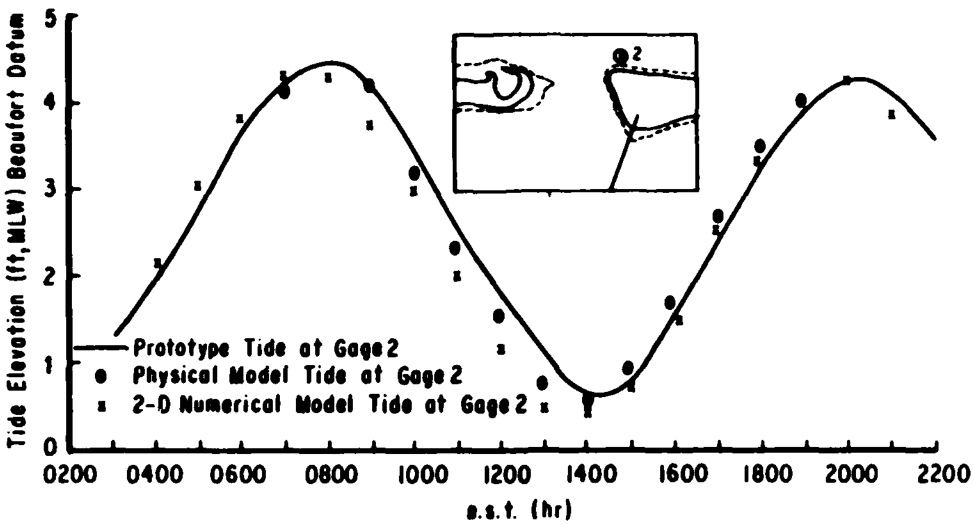


Figure 6. Observed and predicted tides at gage 2 on 12 September 1969 (Harris and Bodine, 1977).

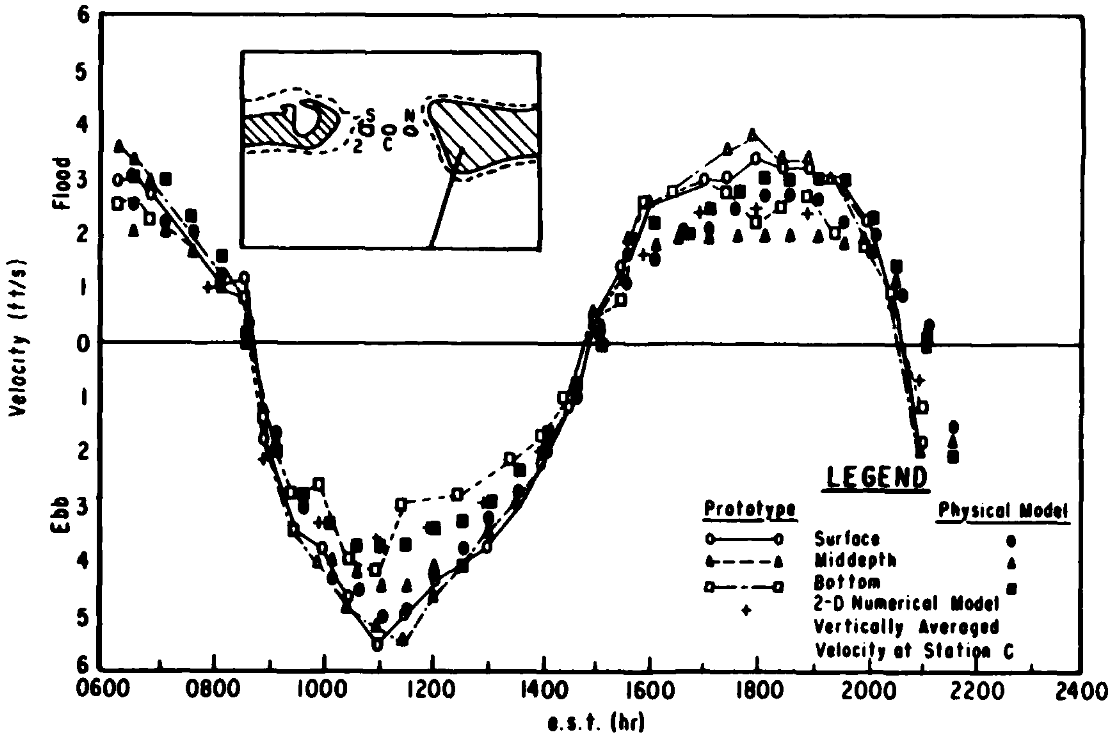


Figure 7. Prototype and predicted velocity for Masonboro Inlet at range 2, station C, on 12 September 1969.

3. Verification.

The physical model was remodeled to the 1974 condition with the friction strips kept in the same orientation as determined from calibration. It was assumed that the frictional characteristics of the two conditions were similar. The model was operated using the observed ocean tide for the verification condition. Typical results for the prediction of tides and currents are shown in Figures 8 and 9. The detailed predictions of the model and corresponding prototype data are given in the Appendix.

As in the calibration runs, waves were subsequently introduced into the model. For the verification condition, prototype waves 1.3 feet (0.4 meter) high with a period of 7 seconds were observed breaking at an angle of S. 25°E. at a pier 2.3 miles (3.7 kilometers) north of the inlet. Waves in the model caused a slight general increase in bay water levels; a similar observation was made during calibration. For both tidal elevations and velocities during verification, waves did not significantly change the overall accuracy of model predictions, except at the throat velocity range where predictions improved with waves. Seabergh and Mason (1975) attributed this lack of effect to small wave heights and decreased depths over the ebb tidal delta. However, improvement in the agreement of physical model dye streak patterns occurred when wind waves were included in comparison to the nonwave conditions. Detailed statistics summarizing model agreement for calibration and verification are presented in Section V of this report.

## III. TWO-DIMENSIONAL NUMERICAL MODEL

1. Theory and Operation.

The two-dimensional numerical model used in this study was developed and calibrated by Masch, Brandes, and Reagan (1977) under contract to the U.S. Army

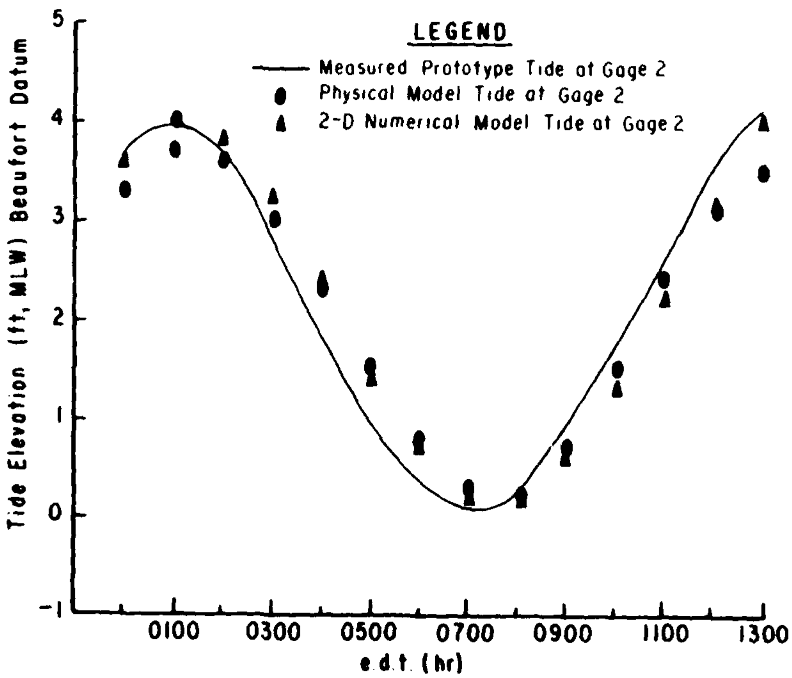


Figure 8. Observed and predicted tides at gage 2 on 25 July 1974.

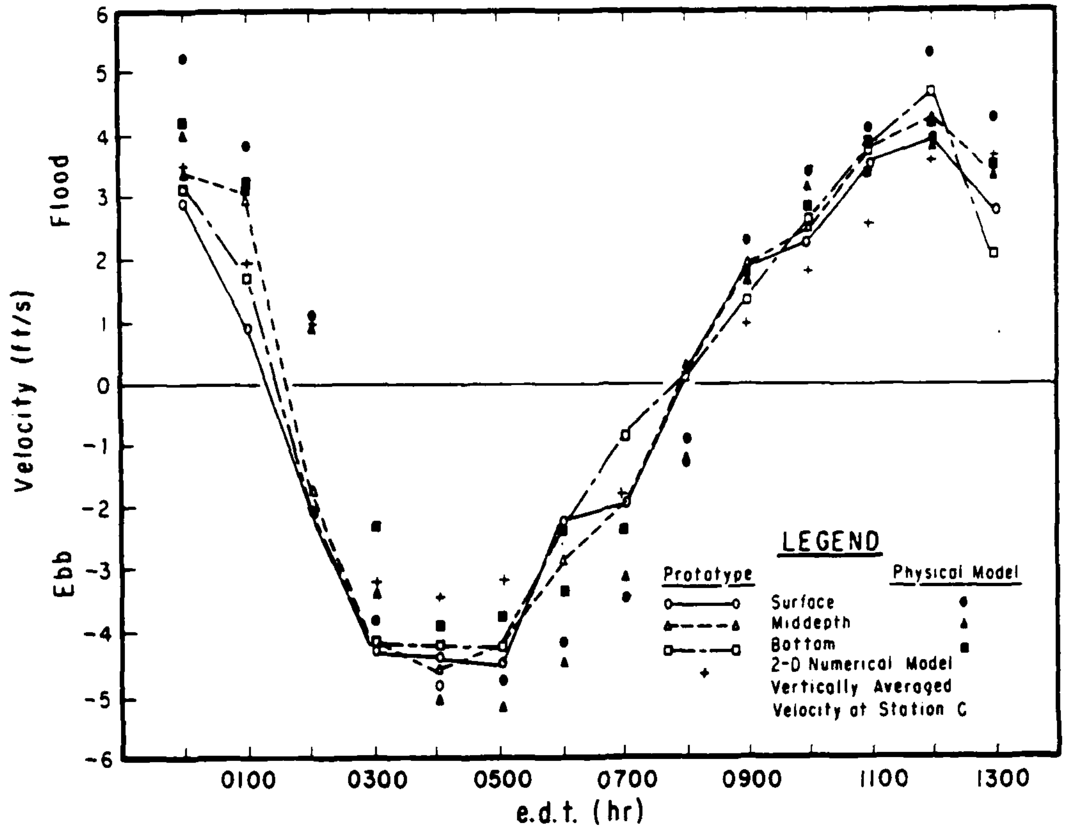


Figure 9. Observed and predicted velocities for Masonboro Inlet at range 2, station C on 25 July 1974.

Coastal Engineering Research Center (CERC). The model is an extension of the storm surge model of Reid and Bodine (1968). It uses a time-centered explicit finite-difference scheme to solve the equations of fluid motion and continuity for water elevations and horizontal transports at selected points. Major assumptions used in developing the model equations are that the fluid is well mixed, the vertical pressure distribution is nearly hydrostatic, and vertical accelerations of the fluid can be neglected. Model features include simulation of alternate flooding and recession of water over mudflats during a tidal cycle, flow over exposed and sumberged barriers, and spatial variation of Manning's friction coefficient. Other effects modeled include Coriolis acceleration, wind stress, and advection of momentum. The model can be driven at boundaries by either specification of nonsinusoidal tides or flow across the boundaries. The vertically averaged velocities are then easily obtained by dividing the magnitude of the horizontal transport per unit width by the total depth at the desired location.

To reduce computer costs, the flow simulation was performed in two steps. The first simulation was a coarse grid model which encompassed a 2.6- by 3.5-mile (4.2 by 5.6 kilometer) area at a spatial resolution of 1,200 feet (366 meters) as shown in Figure 10. The explicit stability criteria for this resolution required a computational time step of 20 seconds. The model was driven by water level variations specified along ocean and channel boundaries.

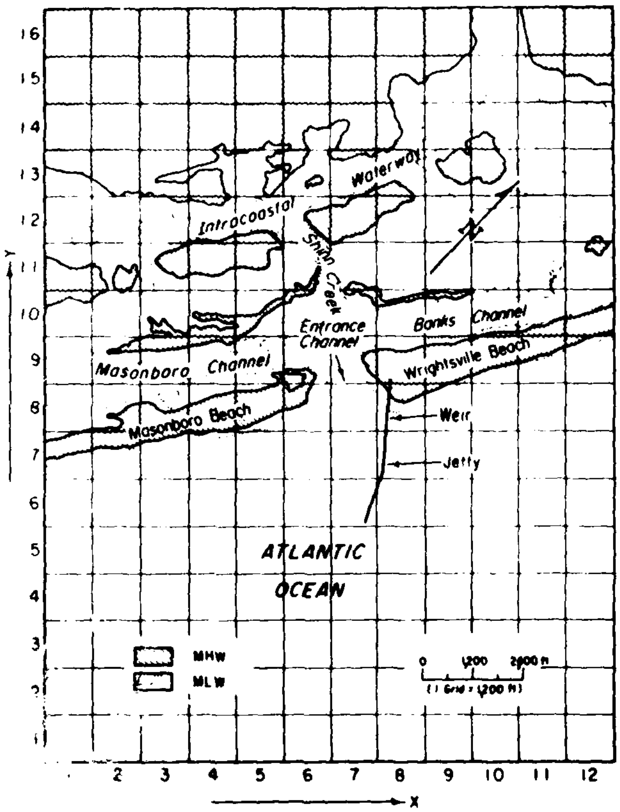


Figure 10. Areal extent of coarse grid, Masonboro Inlet, North Carolina (Masch, Brandes, and Reagan, 1977).

The second simulation was a fine grid model which encompassed a 1.6- by 1.8-mile (2.6 by 2.9 kilometers) area within the area modeled by the first simulation. The spatial resolution of 300 feet (91 meters) used in this model, illustrated in Figure 11, required a time step of 5 seconds for stable computations. The fine grid model was driven by flows retained in the coarse grid model along the common boundary between the two models. Velocity predictions were taken from the fine grid model. This approach allowed resolution of flows of interest without modeling the entire study area at the smaller space and time increments. The increase in speed and the decrease in cost per operation of computers in recent years make this approach less desirable because of the lack of interaction of the models and the increased data handling involved. In 1971, when this study was initiated, the approach taken was considered an attractive means for obtaining sufficiently accurate results at low cost. Additional details of the model including listings and documentation are provided in Masch, Brandes, and Reagan (1977).

2. Calibration.

The coarse grid model did not simulate the entire area containing the tidal prism as was done in the physical model. Instead, water level time histories were applied to channels where flow continued beyond model boundaries. The

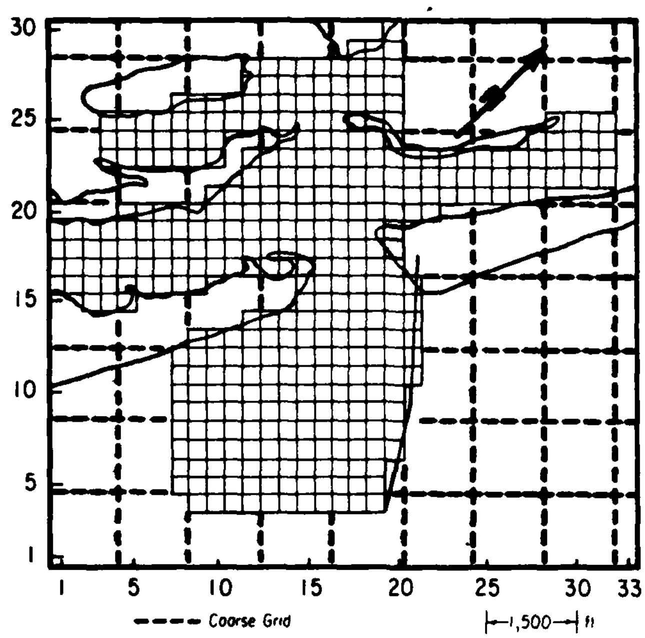


Figure 11. Fine grid configuration used to describe September 1969 conditions (Masch, Brandes, and Reagan, 1977).

procedure of developing the proper tides at these boundaries is somewhat empirical since adjustments of the amplitudes and phases of these boundary tides are part of the calibration process.

The initial values of bottom friction and barrier discharge coefficients were based on empirical data and formulas and on the modeler's experience with similar study areas. Final values were determined by trial-and-error adjustments between model runs to obtain the closest agreement practical between model predictions and prototype data. A constant wind stress in space and time was also included to model the effect of a 4-knot (7.4 kilometer) wind from the northeast.

Typical calibration results for water levels and vertically averaged velocities predicted by the two-dimensional numerical model are shown in Figures 6 and 7. Detailed presentation of the calibration results can be found in Masch, Brandes, and Reagan (1977) and Harris and Bodine (1977).

3. <u>Verification</u>.

Several changes were necessary to prepare the numerical model for the verification runs. Bathymetry and boundary geometry were modified based on the 1974 prototype survey and dredging records. Bottom friction coefficients were revised using a depth-dependent relation,

$$n = 0.0377 + 0.000667\,z, \quad z \leq -4 \text{ feet MSL}$$

$$n = 0.0550 + 0.005\,z, \quad -4 \leq z \leq 0 \text{ foot MSL}$$

$$n = 0.0550 + 0.005\,z, \quad z > 0 \text{ foot MSL}$$

suggested by Masch, Brandes, and Reagan (1977) in their calibration of the Masonboro Inlet model. The tidal forcing boundary conditions at bay channels for the coarse grid model were obtained by modifying the ocean tide given for the new condition in phase and amplitude as determined from calibration. Submerged and overtopped barrier coefficients were left unchanged. Typical verification results are shown in Figures 8 and 9. The appendix gives the detailed predictions for the model as well as the corresponding prototype data for tides and currents.

## IV. SPATIALLY INTEGRATED NUMERICAL MODEL

### 1. Theory and Operation.

The spatially integrated numerical model, developed by Seelig, Harris, and Herchenroder (1977), is a simple numerical model that solves the area-averaged momentum equation for the inlet and the continuity equation for the inlet-bay system. Major assumptions of the model include: (a) the bay water surface remains horizontal as it varies in elevation over a tidal cycle; (b) storage of water in the inlet is negligible compared to that in the bay; and (c) wind stress, radiation stress, and Coriolis acceleration can be neglected. A flow net, such as that shown in Figure 12, is constructed through which the model assumes the flow is confined between the ocean and bay. The depth data, friction coefficients, and the flow net determine a first-order differential equation which is solved for the mean velocity in the inlet minimum cross section by a Runge-Kutta-Gill technique. Flow is distributed throughout the flow net so that friction is minimized in each channel. The model predicts the time-dependent bay level, inlet discharge, and mean velocity in the minimum cross section. Additional details about the model and its application are provided in Seelig, Harris, and Herchenroder (1977), and Seelig and Sorensen (1977).

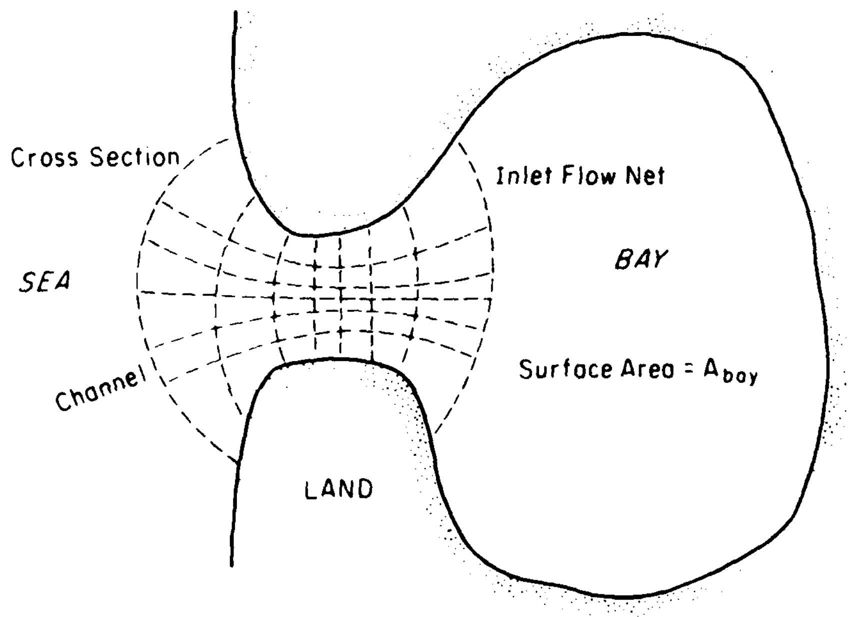


Figure 12. Spatially integrated numerical model typical inlet flow net (Seelig, 1977).

2. Calibration.

The flow net constructed for Masonboro Inlet consisted of four channels and seven cross sections (see Fig. 13). Although the prototype bay area was not well defined, it was estimated by dividing the measured tidal prism by the mean bay tidal range at gages 3, 4, and 5 shown in Figure 13. This gave a surface area of $1.8 \times 10^8$ square feet ($1.7 \times 10^7$ square meters) MSL. A linear increase in bay area of $3.6 \times 10^7$ square feet per foot ($3.3 \times 10^6$ square meters per meter) increase in bay level, used to model flooding of tidal mudflats, was estimated from hydrographic charts. Manning's bottom friction coefficients were assigned using the depth-dependent relation used by Masch, Brandes, and Reagan (1977) in their calibration of the two-dimensional numerical model. Results of the model calibration are shown in Figures 14 and 15.

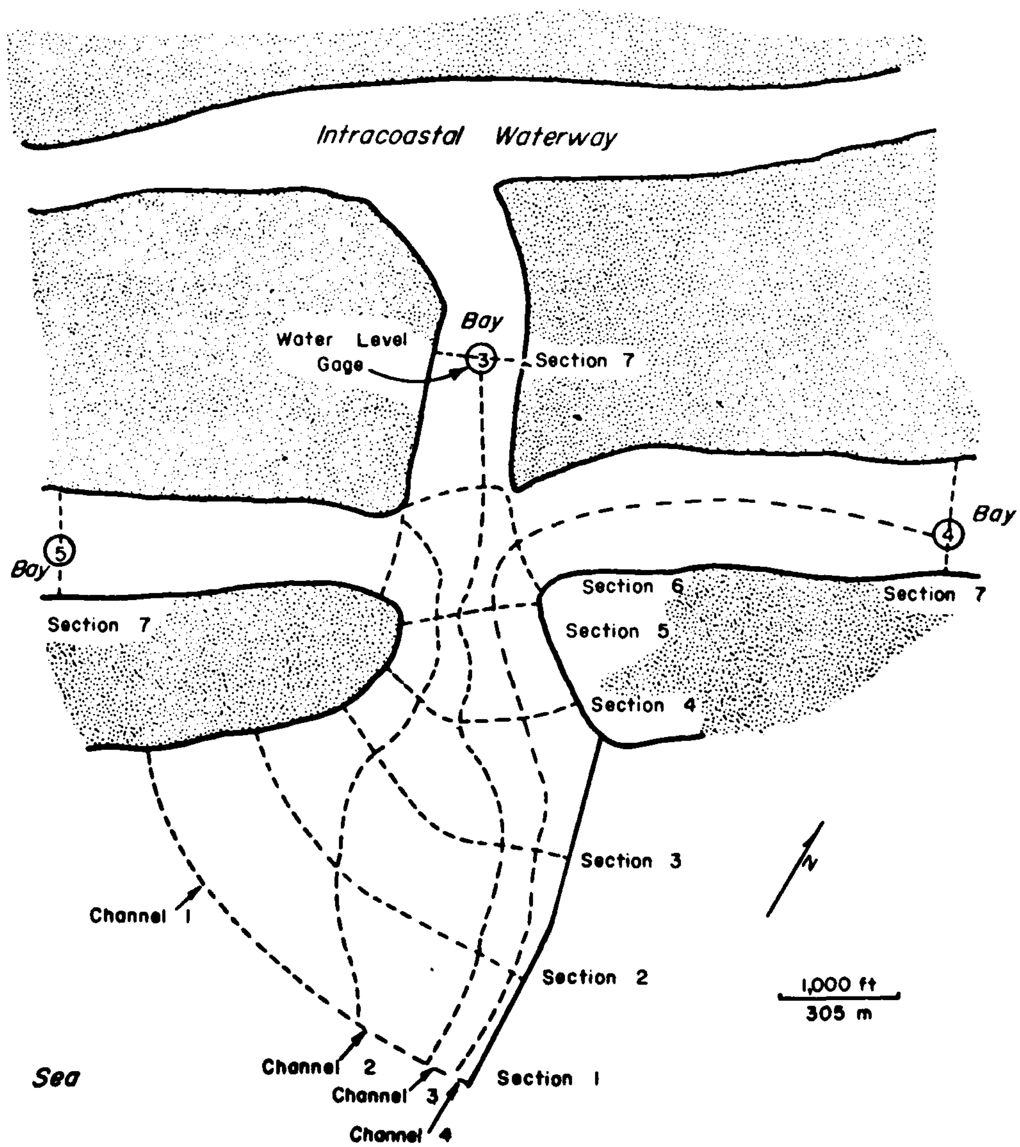


Figure 13. Spatially integrated model grid system for Masonboro Inlet, 1969 (Seelig, Harris, and Herchenroder, 1977).

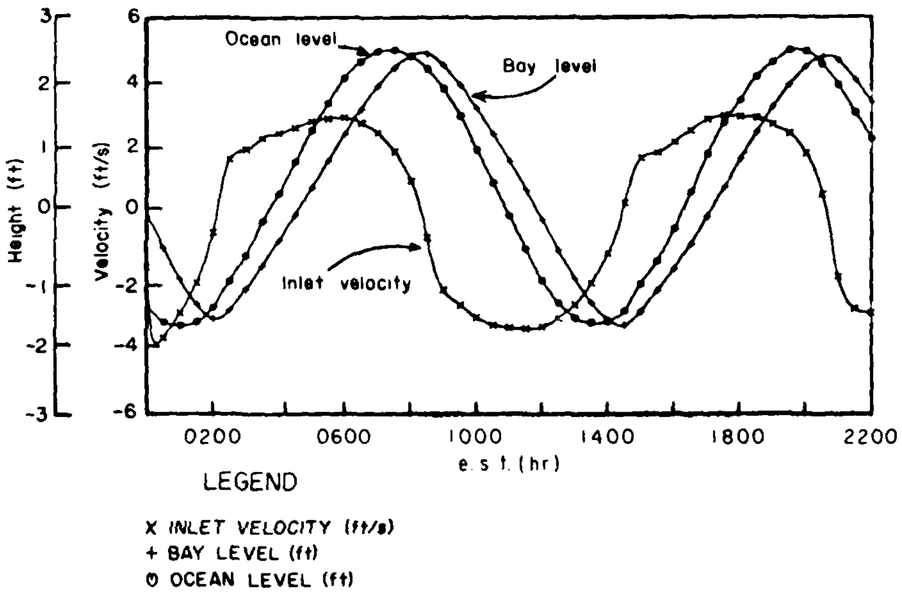


Figure 14. Masonboro Inlet hydraulics, predicted 12 September 1969 by spatially integrated numerical model (from Seelig, Harris, and Herchenroder, 1977).

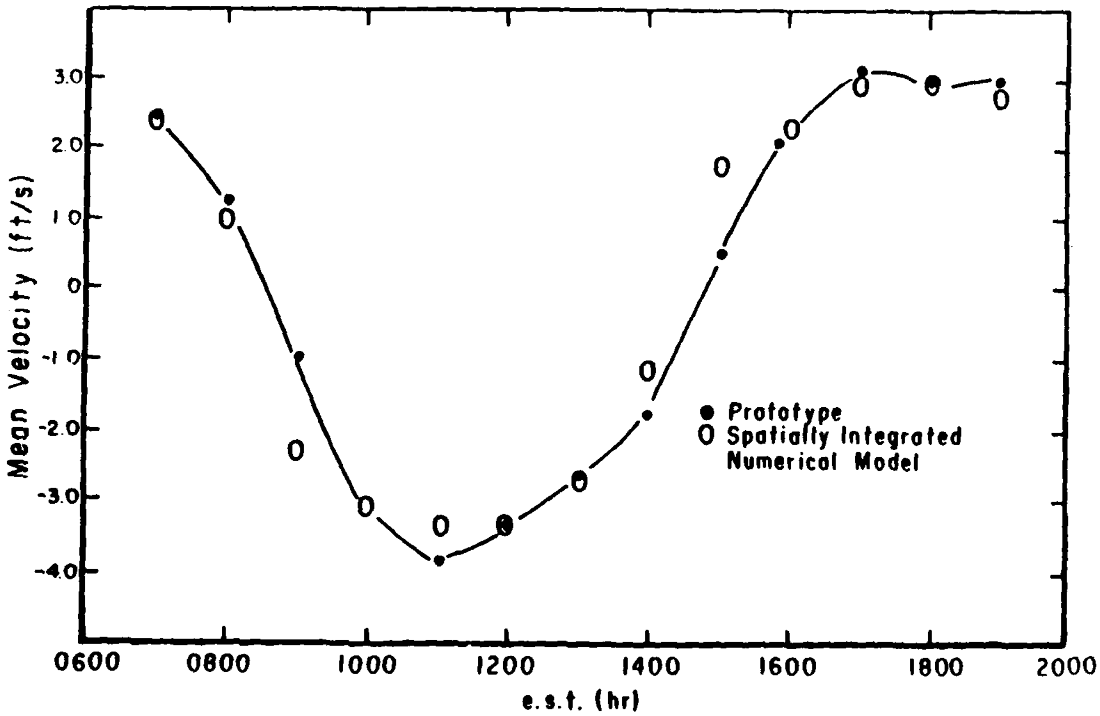


Figure 15. Prototype and mean velocity in the throat of Masonboro Inlet, predicted 12 September 1969 by the spatially integrated numerical model (Seelig, Harris, and Herchenroder, 1977).

3. Verification.

The procedures used for modifying the spatially integrated model to the new condition were similar to those followed for the two-dimensional numerical model. Bathymetry and geometry changes were incorporated into a revised flow net. The flow net was redrawn to increase the width of the grid channel which included the ebb tidal delta. This widening was required to keep streamlines within the grid channel as much as possible since the ebb tidal delta had grown between the 1969 and 1974 conditions.

New Manning's n bottom friction coefficients were assigned based on Masch, Brandes, and Reagan's (1977) relation. The bay area was assumed to vary with bay level in the same manner as for the calibration condition. The model was then driven by the new ocean tide. The verification predictions for the average bay level response and mean velocity are shown in Figures 16 and 17, respectively.

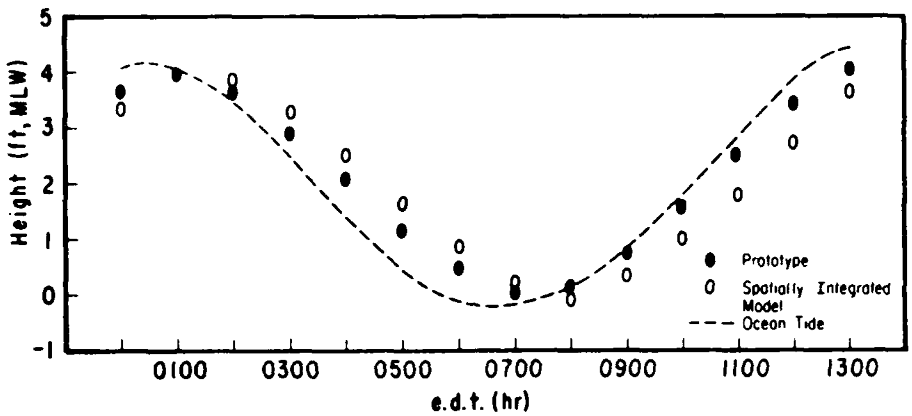


Figure 16. Prototype and spatially integrated model prediction of the mean bay level in Masonboro Inlet on 25 July 1974.

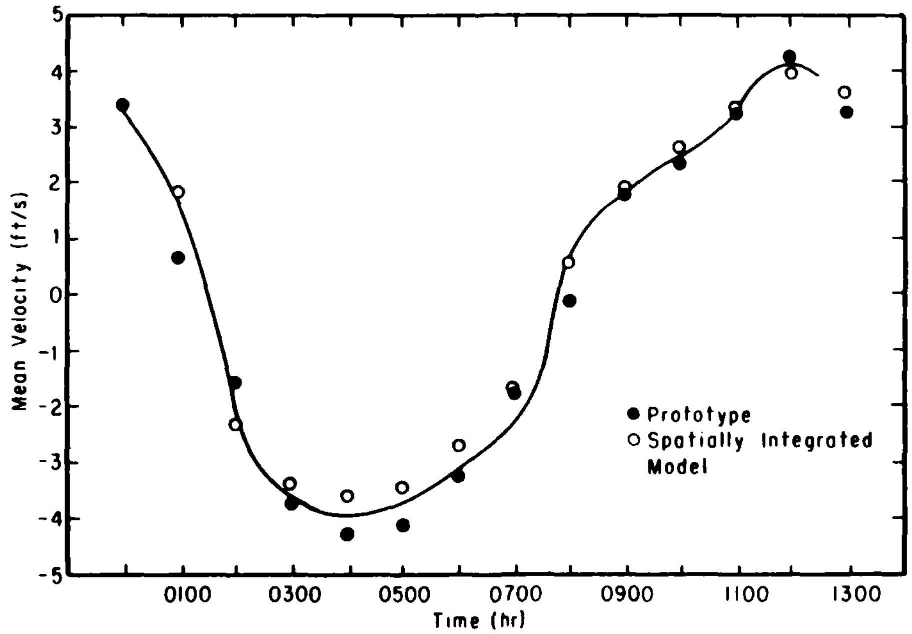


Figure 17. Prototype and spatially integrated model prediction of mean velocity in Masonboro Inlet on 25 July 1974.

## V. COMPARISON OF MODEL RESULTS

### 1. Comparison Statistics.

The following statistics, suggested by Harris and Bodine (1977) and Seabergh and Mason (1975), were useful in comparing the models:

The root-mean-square error, rms$\Delta$,

$$\mathrm{rms}\Delta = \left[\frac{1}{N}\sum (Y - X)^2\right]^{1/2}$$

and the average absolute error, $\overline{|\Delta|}$,

$$\overline{|\Delta|} = \frac{1}{N}\sum |Y - X|$$

where $Y$ is the model value, $X$ the prototype value, and $N$ the number of values compared.

The rms error was used to compare model and prototype records of velocity and water levels in time; the average error was used to compare model and prototype extreme values of all gages collectively. The extreme values correspond to high and low water levels and peak flood and ebb velocities.

The rms error is a commonly used statistic, but for small numbers of points it gives more weight to large errors. The average absolute error was preferred for comparing extreme value errors because it weighs each error equally in the average. When large numbers of points are involved, there is generally no appreciable difference in the two statistics.

The prototype and model extreme values compared did not necessarily coincide in time if there was a lag in the response of the model compared to the prototype. It is assumed here that these statistics are sufficient to evaluate overall model accuracy even though the statistics provide no separation of errors due to amplitude and phase differences between model and prototype.

### 2. Comparison of Mean Inlet Velocities and Average Bay Levels.

a. Calibration. Spatial arithmetic average of instantaneous bay water levels were first calculated for the prototype, the physical model, and the two-dimensional numerical model in order to make comparisons with the spatially integrated model. Simple averages of velocities in the minimum cross section of the prototype and the physical model were also made to obtain mean velocities. Mean velocities in the two-dimensional numerical model were calculated from the total horizontal transport through the model cross section.

The physical and two-dimensional models produced almost identical calibration results for average bay levels (Table 1). The spatially integrated model overpredicted the bay range by 8 percent, causing the bay range to be nearly equal to the ocean tidal range. This did not occur in the prototype, the physical model, or the two-dimensional numerical model.

Table 1. Comparison of calibration average bay level data.

| | Prototype (ft, MLW) | Physical model (ft, MLW) | 2-D numerical model (ft, MLW) | Spatially integrated model (ft, MLW) | Ocean tide (ft, MLW) |
|---|---|---|---|---|---|
| High water | 4.42 | 4.31 | 4.30 | 4.3 | 4.43 |
| Low water | 0.66 | 0.59 | 0.57 | 0.2 | 0.28 |
| Tidal range | 3.76 | 3.72 | 3.73 | 4.1 | 4.15 |
| rms$\Delta$ | | 0.18 | 0.10 | 0.34 | |

The rms error for the spatially integrated model was also higher. Although predicting slightly different peak mean velocities, as shown in Table 2, the physical and two-dimensional numerical models were equally well calibrated overall for mean velocities as indicated by close values of rms$\Delta$. The spatially integrated model reproduced peak mean velocities better than the other models but the overall calibration was questionable. However, the accuracy of the data and the averaging techniques used may make differences in velocity rms$\Delta$ on the order of 0.1 to 0.2 foot (3 to 6 centimeters) per second statistically insignificant.

Table 2. Comparison of calibration inlet mean velocity data.

| | Prototype (ft/s) | Physical model (ft/s) | 2-D numerical model (ft/s) | Spatially integrated model (ft/s) |
|---|---|---|---|---|
| Peak flood | 3.04 | 3.72 | 2.94 | 3.0 |
| Peak ebb | 3.58 | 3.27 | 3.08 | 3.3 |
| rms$\Delta$ | | 0.30 | 0.34 | 0.52 |

b. Verification. The two-dimensional numerical model produced the best overall verification predictions of average bay levels (Table 3). This model predicted the most reliable high water level and the bay tidal range. The physical model had an rms error slightly smaller than the two-dimensional numerical model but the error was not significant. The spatially integrated model again overestimated the bay level tidal range and had the largest rms error for prediction of average bay levels. The significance of the rms error difference is uncertain because of the variability in the estimate of prototype average bay levels. Predictions of the low water bay elevation were about equal for all models.

Table 3. Comparison of verification average bay level data.

| | Prototype (ft, MLW) | Physical model without waves (ft, MLW) | Physical model with waves (ft, MLW) | 2-D numerical model (ft, MLW) | Spatially integrated model (ft, MLW) | Ocean tide (ft, MLW) |
|---|---|---|---|---|---|---|
| High water | 3.96 | 3.72 | 3.72 | 3.92 | 4.07 | 4.17 |
| Low water | 0.07 | 0.24 | 0.28 | 0.23 | 0.09 | 0.22 |
| Tidal range | 3.89 | 3.48 | 3.44 | 3.69 | 4.16 | 4.39 |
| rms$\Delta$ | | 0.27 | 0.30 | 0.30 | 0.42 | |

The spatially integrated model produced the best overall verification predictions for inlet mean velocities (Table 4). The physical model predicted the most reliable peak ebb velocity but the model's rms error was significantly larger than that of the spatially integrated model. The two-dimensional numerical model predictions for peak velocities were significantly less than the prototype values. This model's rms error was also significantly larger than that of the spatially integrated model but smaller than that of the physical model. It is interesting to note that both the rms errors for water level and velocity predictions from the spatially integrated model did not increase significantly from their values for the calibration conditions.

Table 4. Comparison of verification mean velocity data.

| | Prototype | Physical model | | 2-D numerical model | Spatially integrated model |
|---|---|---|---|---|---|
| | | Without waves | With waves | | |
| | (ft/s) | (ft/s) | (ft/s) | (ft/s) | (ft/s) |
| Peak flood | 4.19 | 3.84 | 3.74 | 3.39 | 4.11 |
| Peak ebb | 4.26 | 3.90 | 3.87 | 3.09 | 3.79 |
| rmsΔ | | 1.16 | 1.02 | 0.80 | 0.54 |

The statistics in Tables 3 and 4 summarize the effect of modeling waves in the physical model. The inclusion of waves in the model improved the high water elevations at three gages by 0.1 foot, produced no change at two gages, and decreased the reliability of the predicted elevation by 0.1 foot at one gage. Low water elevations were improved at one gage by 0.1 foot, less reliable at two other gages, and were unchanged at the remaining gages. Velocities were consistently improved only at the inlet throat when waves were included.

3. Comparison of Two-Dimensional Numerical Model and Physical Model.

a. Calibration. Harris and Bodine (1977) compared the calibration data for the physical and two-dimensional numerical model and concluded that neither of the models appeared significantly superior to the other for reproducing tides at all tidal stations. Also they found that each model reproduced velocities equally well within the accuracy of the limited data used in their analysis. They cautioned that the models may not have been optimally calibrated; therefore, significant differences may have appeared if this had been done. The calibration statistics summarized in Table 5 are composite values based on all predictions of water levels and velocities at all gages and ranges.

Table 5. Comparison summary of calibration data.

| | Physical model | 2-D numerical model |
|---|---|---|
| High water $\overline{\lvert\Delta\rvert}$(ft) | 0.13 | 0.15 |
| Low water $\overline{\lvert\Delta\rvert}$(ft) | 0.11 | 0.11 |
| Tidal range $\overline{\lvert\Delta\rvert}$(ft) | 0.13 | 0.15 |
| Water level rmsΔ(ft) | 0.21 | 0.21 |
| Peak flood $\overline{\lvert\Delta\rvert}$(ft/s) | 0.40 | 0.38 |
| Peak ebb $\overline{\lvert\Delta\rvert}$(ft/s) | 0.44 | 0.66 |
| Velocity rmsΔ(ft/s) | 0.49 | 0.50 |

b. <u>Verification</u>. Tables 6 to 9 list verification statistics of the models for predicting water levels. Although the two-dimensional numerical model gave significantly better predictions of high water levels and tidal ranges, the overall rms errors for the models were nearly equal.

Table 6. Comparison of verification high water level data.

| Gage | Prototype (ft, MLW) | Physical model: Without waves (ft, MLW) | Physical model: With waves (ft, MLW) | 2-D numerical model (ft, MLW) |
|---|---|---|---|---|
| 1 | 4.30 | 4.1 | 4.2 | 4.04 |
| 2 | 4.00 | 3.7 | 3.7 | 3.99 |
| 3 | 4.01 | 3.8 | 3.8 | 3.95 |
| 4 | 3.87 | 3.6 | 3.7 | 4.00 |
| 5 | 3.93 | 3.7 | 3.7 | 3.83 |
| 5I | 4.00 | 3.8 | 3.7 | 3.97 |
| $\overline{\lvert\Delta\rvert}$, | | 0.24 | 0.22 | 0.10 |

Table 7. Comparison of verification low water level data.

| Gage | Prototype (ft, MLW) | Physical model: Without waves (ft, MLW) | Physical model: With waves (ft, MLW) | 2-D numerical model (ft, MLW) |
|---|---|---|---|---|
| 1 | 0.04 | -0.1 | 0.1 | -0.10 |
| 2 | 0.10 | 0.2 | 0.3 | 0.19 |
| 3 | 0.08 | 0.3 | 0.3 | 0.28 |
| 4 | 0.02 | 0.2 | 0.3 | 0.21 |
| 5 | 0.12 | 0.3 | 0.3 | 0.29 |
| 5I | 0.05 | 0.2 | 0.2 | 0.09 |
| $\overline{\lvert\Delta\rvert}$, | | 0.16 | 0.18 | 0.14 |

Table 8. Comparison of verification tidal range data.

| Gage | Prototype (ft) | Physical model: Without waves (ft) | Physical model: With waves (ft) | 2-D numerical model (ft) |
|---|---|---|---|---|
| 1 | 4.26 | 4.2 | 4.1 | 4.14 |
| 2 | 3.90 | 3.5 | 3.4 | 3.80 |
| 3 | 3.93 | 3.5 | 3.5 | 3.67 |
| 4 | 3.85 | 3.4 | 3.4 | 3.79 |
| 5 | 3.81 | 3.4 | 3.4 | 3.54 |
| 5I | 3.95 | 3.6 | 3.5 | 3.88 |
| $\overline{\lvert\Delta\rvert}$ | | 0.35 | 0.40 | 0.15 |

Table 9. Comparison of verification water level root-mean-square errors.

| Gage | Physical model | | 2-D numerical model (ft) |
|---|---|---|---|
| | Without waves (ft) | With waves (ft) | |
| 1 | 0.25 | 0.19 | 0.25 |
| 2 | 0.33 | 0.36 | 0.29 |
| 3 | 0.29 | 0.27 | 0.44 |
| 4 | 0.30 | 0.34 | 0.27 |
| 5 | 0.26 | 0.30 | 0.42 |
| 5I | 0.24 | 0.26 | 0.15 |
| Total | 0.28 | 0.29 | 0.32 |

Tables 10, 11, and 12 list verification statistics for vertically averaged velocities. The physical model predictions were slightly better than the numerical model for peak flood velocities and significantly better for peak ebb velocities. However, the numerical model had a lower overall rms error for velocities.

Table 10. Comparison of verification peak flood velocity data.

| Gage | Prototype (ft/s) | Physical model | | 2-D numerical model (ft/s) |
|---|---|---|---|---|
| | | Without waves (ft/s) | With waves (ft/s) | |
| [illegible] | 2.15 | 1.67 | 1.00 | 2.13 |
| [illegible] | 2.60 | [illegible] | [illegible] | 1.77 |
| 1S | [illegible] | 2.43 | 2.32 | 1.48 |
| 2N | [illegible] | 3.05 | 3.70 | 4.23 |
| [illegible] | [illegible] | 4.19 | 4.04 | 3.65 |
| [illegible] | [illegible] | [illegible] | 3.55 | 2.40 |
| 3N | [illegible] | 2.43 | [illegible] | 1.83 |
| [illegible] | [illegible] | [illegible] | [illegible] | [illegible] |
| [illegible] | [illegible] | 1.50 | [illegible] | [illegible] |
| 4E | 1.99 | 1.00 | [illegible] | 1.38 |
| [illegible] | 1.69 | 1.53 | 1.43 | [illegible] |
| 4W | 1.77 | 1.60 | 1.85 | 1.08 |
| 4IE | 1.44 | 1.36 | 1.13 | 1.66 |
| 4IC | 2.53 | 2.33 | 2.27 | 1.69 |
| 4[illegible] | 1.99 | 2.47 | 2.33 | 1.11 |
| 5E | 1.83 | 2.90 | 2.53 | 1.78 |
| 5C | 1.90 | 2.50 | 2.50 | 1.86 |
| 5W | 1.40 | 2.10 | 2.10 | 1.92 |
| 5IE | 1.93 | 2.43 | 2.29 | [illegible] |
| 5IC | 1.99 | 2.30 | 2.34 | 1.42 |
| 5IW | 1.91 | 1.95 | 1.95 | 1.46 |
| [illegible] | | 0.48 | 0.48 | 0.59 |

Table 11. Comparison of verification peak ebb velocities.

| Gage | Prototype (ft/s) | Physical model: Without waves (ft/s) | Physical model: With waves (ft/s) | 2-D numerical model (ft/s) |
|---|---|---|---|---|
| 1N | 4.17 | 3.87 | 3.80 | 3.63 |
| 1C | 4.22 | 3.87 | 3.67 | 3.41 |
| 1S | 3.53 | 3.96 | 4.27 | 2.52 |
| 2N | 3.97 | 2.41 | 2.54 | 3.24 |
| 2C | 4.31 | 4.53 | 4.39 | 3.36 |
| 2S | 4.50 | 4.83 | 4.80 | 2.96 |
| 3N | 2.82 | 2.27 | 2.27 | 1.47 |
| 3C | 2.14 | 2.27 | 2.33 | 1.89 |
| 3S | 1.86 | 2.1[illegible] | 2.47 | 2.13 |
| 4E | 2.11 | 2.65 | 2.30 | 1.26 |
| 4C | 1.71 | 1.87 | 1.57 | 1.48 |
| 4W | 1.22 | 1.65 | 1.30 | 1.19 |
| 4IE | 3.04 | 3.57 | 3.37 | 1.52 |
| 4IC | 2.53 | 2.93 | 3.03 | 1.79 |
| 4IW | 1.43 | 1.29 | 1.37 | 0.99 |
| 5E | 1.94 | 2.60 | 2.40 | 1.05 |
| 5C | 2.12 | 2.55 | 2.45 | 1.23 |
| 5W | 1.32 | 2.30 | 2.30 | 1.38 |
| 5IE | 2.75 | 2.60 | 2.58 | 1.60 |
| 5IC | 2.28 | 2.49 | 2.45 | 1.84 |
| 5IW | 1.72 | 1.20 | 1.40 | 1.52 |
| [illegible] | | 0.44 | 0.41 | 0.71 |

Table 12. Comparison of verification rms errors in velocities.

| Gage | Physical model: Without waves (ft/s) | Physical model: With waves (ft/s) | 2-D numerical model (ft/s) |
|---|---|---|---|
| 1N | 0.77 | 0.98 | 0.39 |
| 1C | 0.82 | 0.76 | 0.69 |
| 1S | 0.95 | 0.77 | 0.93 |
| 2N | 1.18 | 1.06 | 0.56 |
| 2C | 1.57 | 1.46 | 0.92 |
| 2S | 1.43 | 1.33 | 1.23 |
| 3N | 0.37 | 0.33 | 0.85 |
| 3C | 0.44 | 0.48 | 0.56 |
| 3S | 0.54 | 0.67 | 0.47 |
| 4E | 0.72 | 0.77 | 0.50 |
| 4C | 0.50 | 0.48 | 0.33 |
| 4W | 0.56 | 0.64 | 0.39 |
| 4IE | 0.70 | 0.57 | 1.01 |
| 4IC | 0.62 | 0.68 | 0.67 |
| 4IW | 0.37 | 0.25 | 0.68 |
| 5E | 0.85 | 0.83 | 0.55 |
| 5C | 0.75 | 0.70 | 0.48 |
| 5W | 0.81 | 0.66 | 0.39 |
| 5IE | 0.69 | 0.82 | 0.80 |
| 5IC | 1.03 | 1.14 | 0.71 |
| 5IW | 0.69 | 0.63 | 0.39 |
| Total | 0.87 | 0.84 | 0.73 |

Out of six gages the physical model high water elevations lagged the prototype by 0.5 hour at one gage and 1.0 hour at another gage. For the same two gages, the two-dimensional numerical model lagged the prototype by 1.0 hour and at two additional gages it lagged by 0.5 and 0.75 hour. For low water elevations, both the physical and two-dimensional numerical models lagged the prototype by 0.5 hour at four gages. This indicates either there was correspondingly more friction in the two models or the bay area in the models was correspondingly greater than that in the prototype or possibly a combination of the two.

Peak ebb current tended to lag in the physical model an average of about 0.5 hour behind the prototype. However, peak flood current tended to occur 0.5 hour ahead of the prototype. The two-dimensional numerical model tended to lead the prototype slightly in predicting both peak ebb and flood currents. There was significant scatter in the phase differences from which the above trends were extracted. For example, the physical model lagged the prototype by about 2 hours at 3 out of 27 stations in predicting peak ebb velocities. The physical model also led the prototype by 2 hours at one station in predicting the peak flood velocity. The two-dimensional numerical model was no more than 1 hour out of phase with the prototype in predicting the peak ebb velocity at any station, but at one gage the model lagged the prototype by 1.5 hours in predicting peak flood velocity.

The difference in rms errors between the models is probably not overly significant due to inaccuracies in the data and in the approximations made by using simple vertical averages of surface, middepth, and bottom velocities to make comparisons (surface and bottom velocities were actually recorded 1 foot away from those boundaries). A decrease in accuracy of velocity measurements also resulted from considerable fluctuations which were present in the prototype measurements. The survey procedure was only designed to reduce meter error by averaging six velocity measurements taken close together in time. The resulting accuracy of the mean was about ±6 percent primarily due to the velocity fluctuations. Because of this limited accuracy, differences in rms error between models on the order of 0.1 to 0.2 foot per second are not statistically significant.

As mentioned earlier, the introduction of waves in the physical model caused little overall change in model velocities or water levels. Consequently, the overall accuracy of model predictions was essentially unchanged, as documented in Tables 6 to 12.

## VI. SUMMARY AND CONCLUSIONS

Two numerical models and a physical model were calibrated to the condition of Masonboro Inlet, North Carolina, for September 1969. In comparing calibrations, it was found that the two-dimensional numerical model and the physical model reproduced prototype water elevations and vertically averaged velocities equally well. The introduction of waves in the physical model had little effect on the conditions modeled. The spatially integrated model was not as well calibrated overall in reproducing prototype average bay levels and inlet mean velocities as were the other models. However, this model reproduced the peak ebb and flood velocities more reliably. The higher rms errors for the spatially integrated model may be due either to the simplifications made in the model or to a less than optimum choice of flow net or Manning's friction coefficients.

The models were subsequently modified to incorporate changes which occurred in bathymetry between 1969 calibration and 1974 verification conditions. Overall, the physical model and the two-dimensional numerical model predicted water levels equally well, although the latter model did better in predicting tidal ranges and extreme water levels. This is somewhat surprising considering the crude method used to obtain channel boundary conditions in the two-dimensional numerical model. The spatially integrated model again had the highest rms errors for the prediction of water levels. Verification mean velocities in the inlet were predicted significantly better by the spatially integrated model than by the other two models. The rms error of the spatially integrated model for velocities changed little from calibration to verification; a significant increase was shown by the physical and two-dimensional numerical models. The modeling of waves in the physical model had little effect in changing the accuracy of model predictions either for the calibration or verification predictions. Because of the large fluctuation of prototype velocities sampled at any time, the limited data collected, and also the approximations employed in comparing data from dissimilar sources, differences in velocity rms errors less than 0.2 foot per second were not deemed statistically significant. It is within this limited accuracy that the physical model and the two-dimensional numerical model were judged to be comparable.

The changes in bathymetry and subsequent change in flow patterns between 1969 and 1974 conditions were probably not as dramatic a change as that which occurs, for instance, between the flow patterns before and after a jetty is constructed. In that respect, the prediction of the 1974 hydraulic response was not the most difficult test that could have been devised for the models. Further studies of this type should address problems like the aforementioned one which is of considerable engineering interest. The large memory capacity and computational speed of present-day supercomputers allow a greater resolution of flows than was deemed practical when the computer runs used in the study were made. It is the author's opinion that the two-dimensional model results could be improved by modeling the entire effective bay area at a smaller resolution than the 300 feet used in this study. The dual model approach employed by Masch, Brandes, and Reagan (1977) is outdated because of the improvement in computer technology. Also, the physical model results may have been improved by constructing the model in a larger facility to reproduce a larger ocean area, decrease the distortion factor, and eliminate the need to schematize the estuary area to such a large degree. The state-of-the-art in physical and numerical modeling has improved substantially since these studies were initiated.

Cost is an important factor in engineering design, so the costs quoted for the model studies by Harris and Bodine (1977) are repeated below (Table 13) with additional costs that are incurred in actual application to field cases. Since the lumped-parameter model is similar in ease of operation and cost (but not always as accurate) as the spatially integrated model, and the cost figure was readily available, it is given instead of that for the spatially integrated model.

The cost of prototype data acquisition can often be the major cost of a model application. To achieve a given degree of accuracy the amount and the cost of data are independent of the model chosen (assuming the model is general enough to take advantage of detailed prototype survey data). Consequently, when the total costs of a study are considered, the cost differences between

Table 13. Costs of model studies.

| | Numerical or physical model construction and application | Bathymetric survey | Acquisition and analysis of prototype data |
|---|---|---|---|
| Lumped-parameter model (Huval and Wintergerst, 1977) | $5,000 | $30,000-100,000 | $60,00-300,000 |
| Two-dimensional numerical model (Masch, Brandes, and Reagan, 1977) | $46,035 | $30,000-100,000 | $60,000-300,000 |
| Fixed-bed physical model (Sager and Seabergh, 1977) | $225,000 | $30,000-100,000 | $60,000-300,000 |

[1]Some of the data in the upper limit of this cost estimate are not justified, considering the simplistic representation of inlets and estuaries made by the lump-parameter and spatially integrated models.

models may be of minor importance. In preliminary planning studies where little or no prototype data are available, the spatially integrated or two-dimensional numerical models would be appropriate.

Cost should not be the only determining factor, however, since physical models are more suitable for certain studies than currently available numerical models and visa versa. Rapid improvements are being made in both physical and numerical modeling technology.

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APPENDIX

DETAILED MODEL AND PROTOTYPE
DATA PREDICTIONS

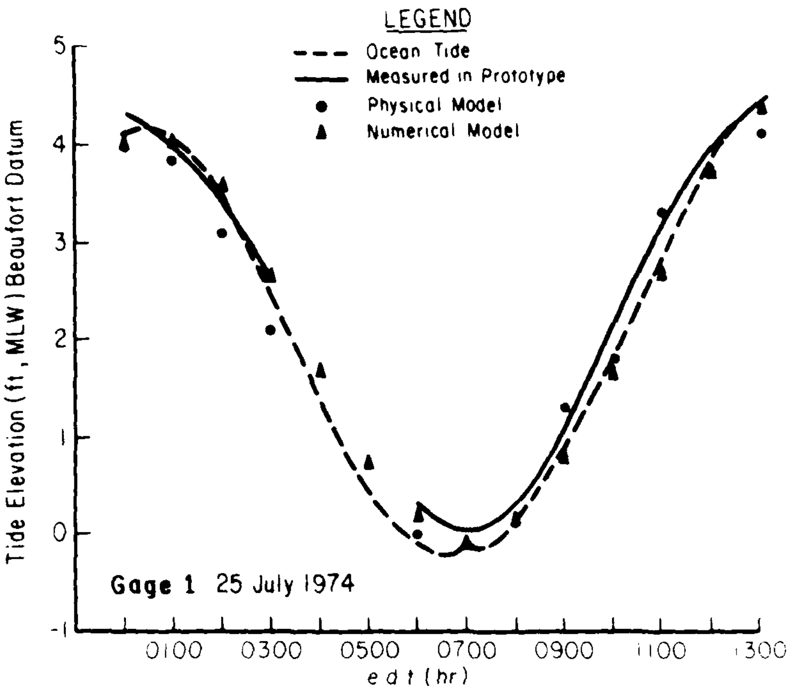


Figure A-1. Observed and predicted tides at gage 1.

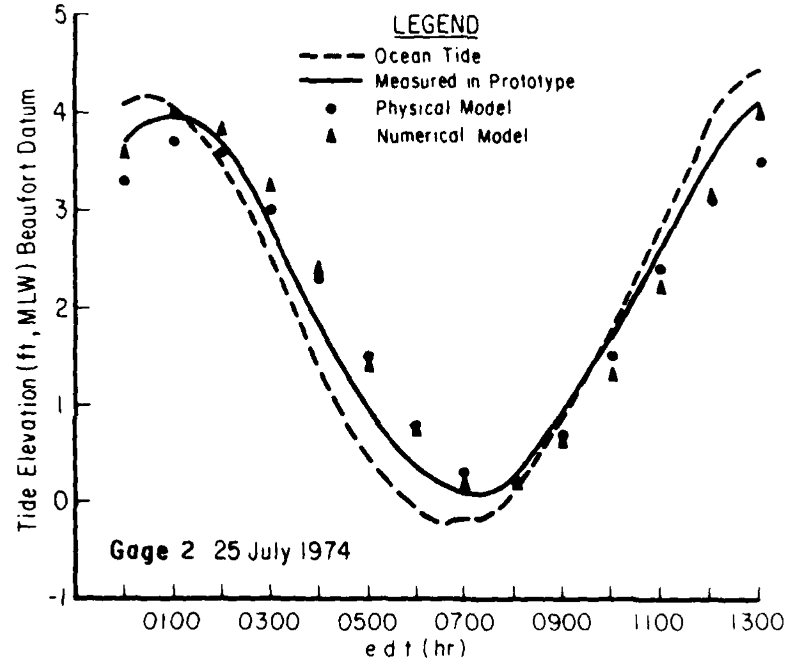


Figure A-2. Observed and predicted tides at gage 2.

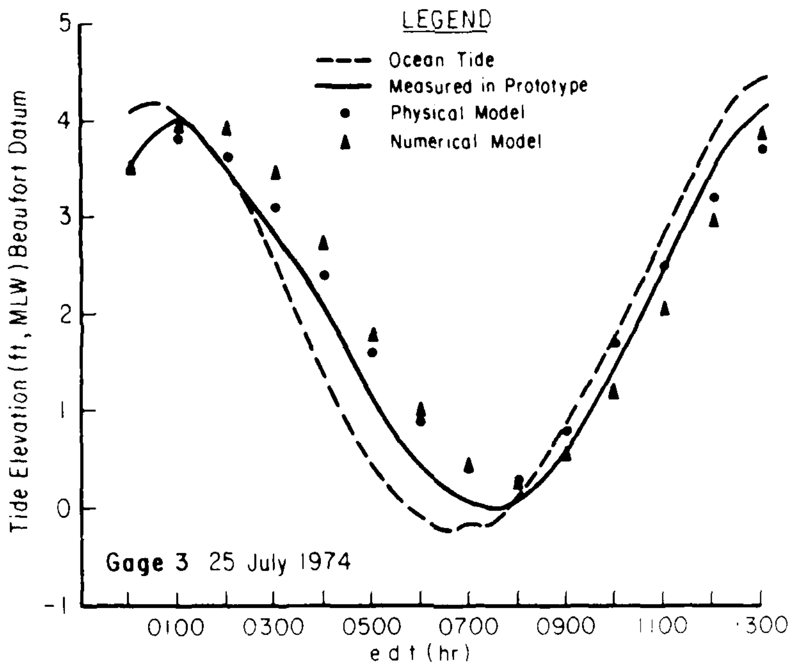


Figure A-3. Observed and predicted tides at gage 3.

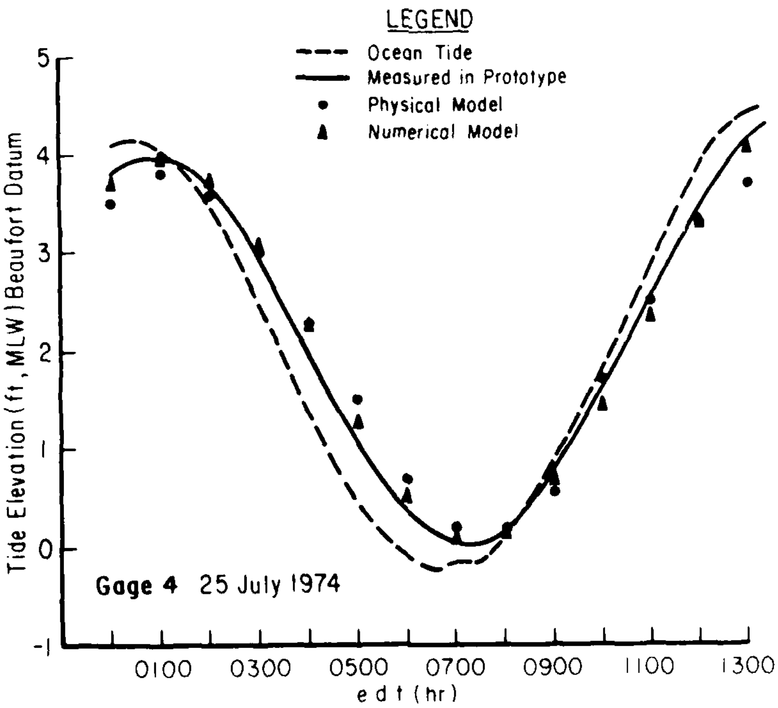


Figure A-4. Observed and predicted tides at gage 4.

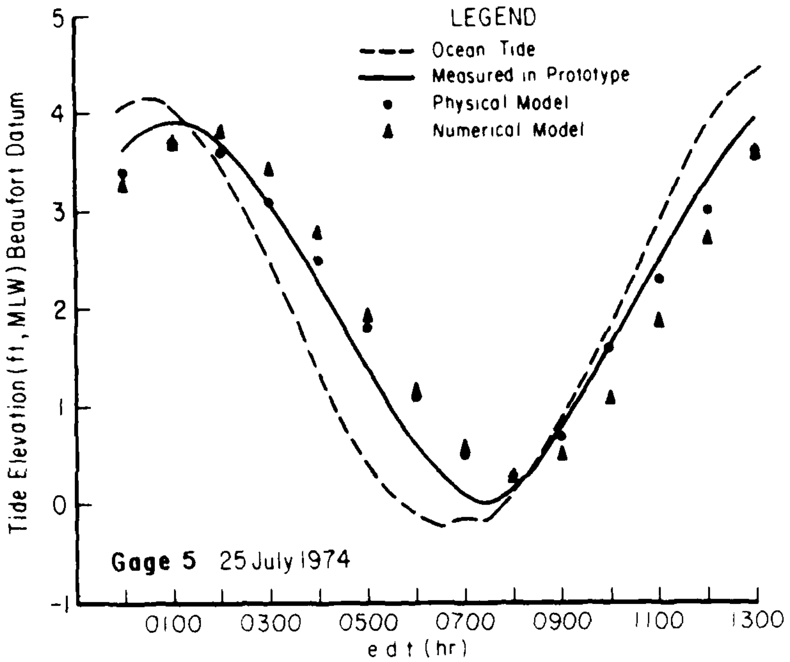


Figure A-5. Observed and predicted tides at gage 5.

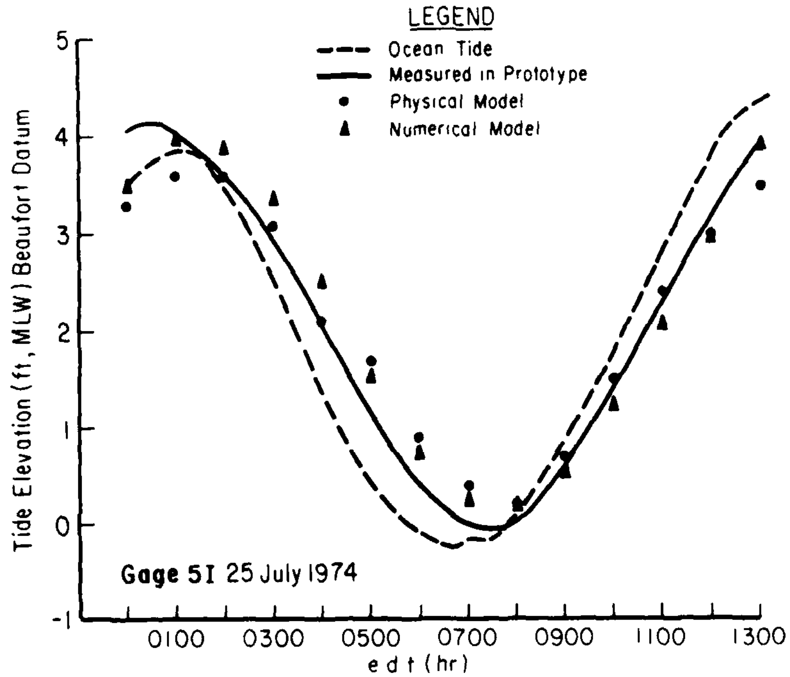


Figure A-6. Observed and predicted tides at gage 5I.

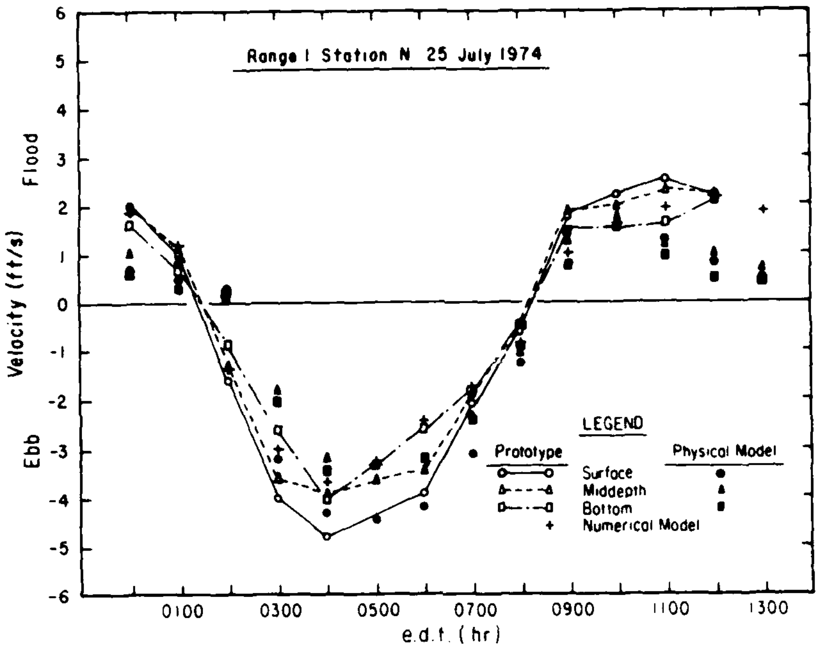


Figure A-7. Observed and predicted velocities at range 1, station N.

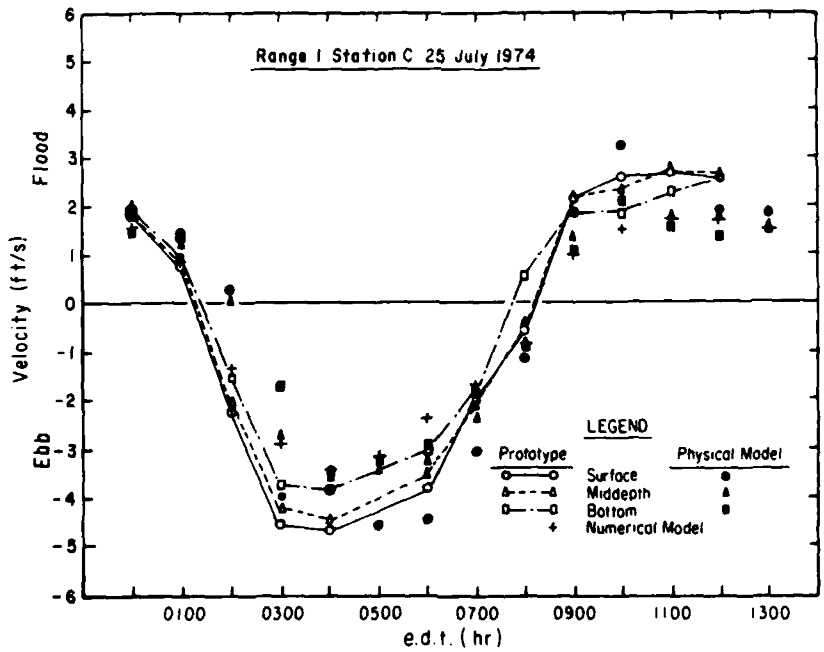


Figure A-8. Observed and predicted velocities at range 1, station C.

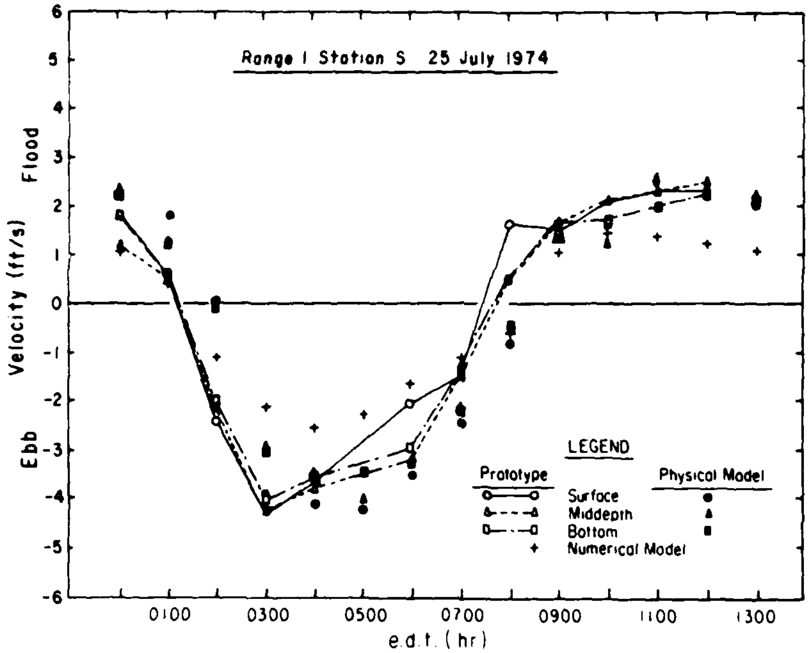


Figure A-9. Observed and predicted velocities at range 1, station S.

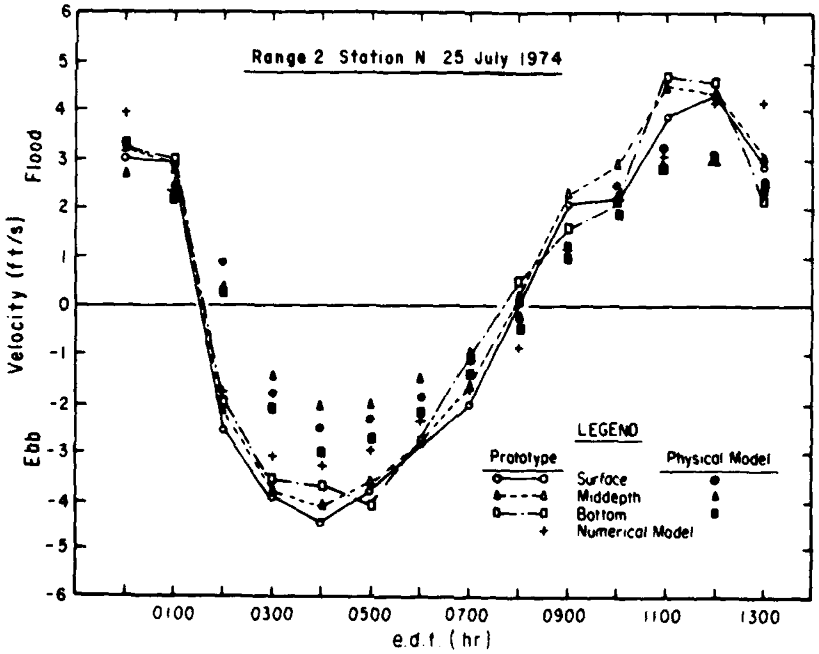


Figure A-10. Observed and predicted velocities at range 2, station N.

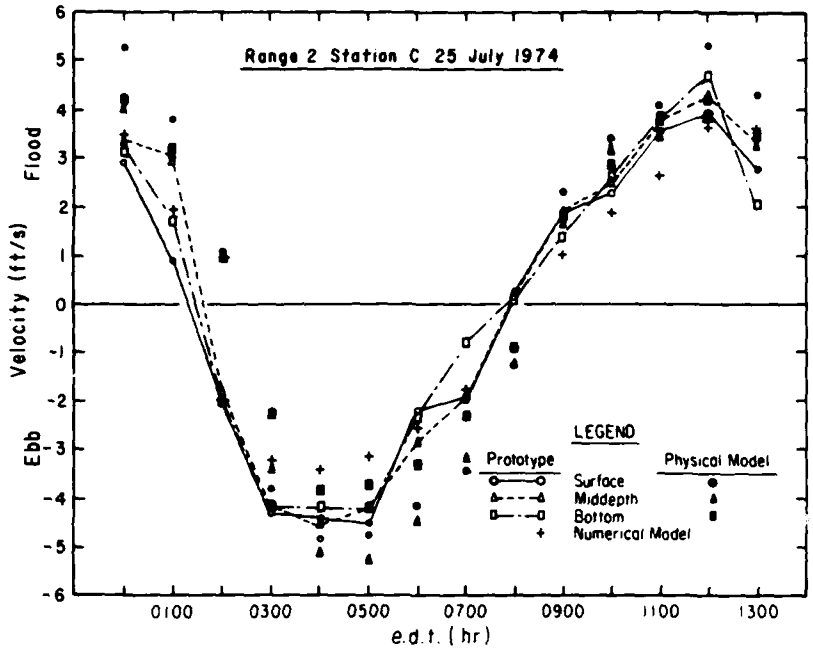


Figure A-11. Observed and predicted velocities at range 2, station C.

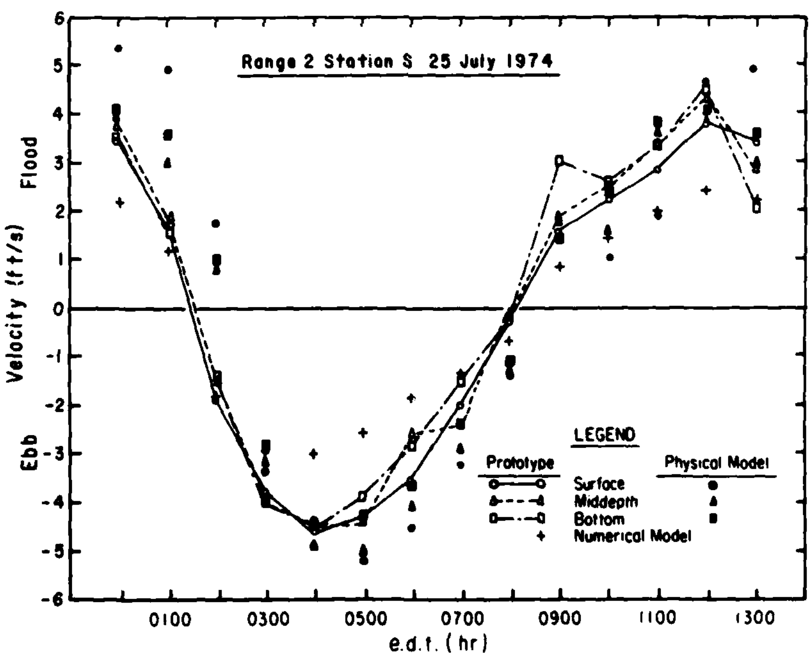


Figure A-12. Observed and predicted velocities at range 2, station S.

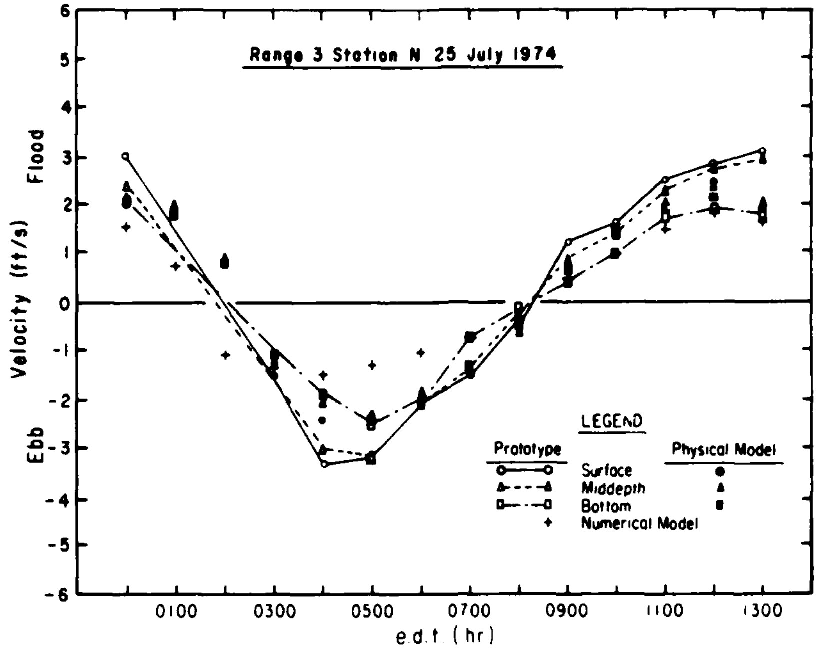


Figure A-13. Observed and predicted velocities at range 3, station N.

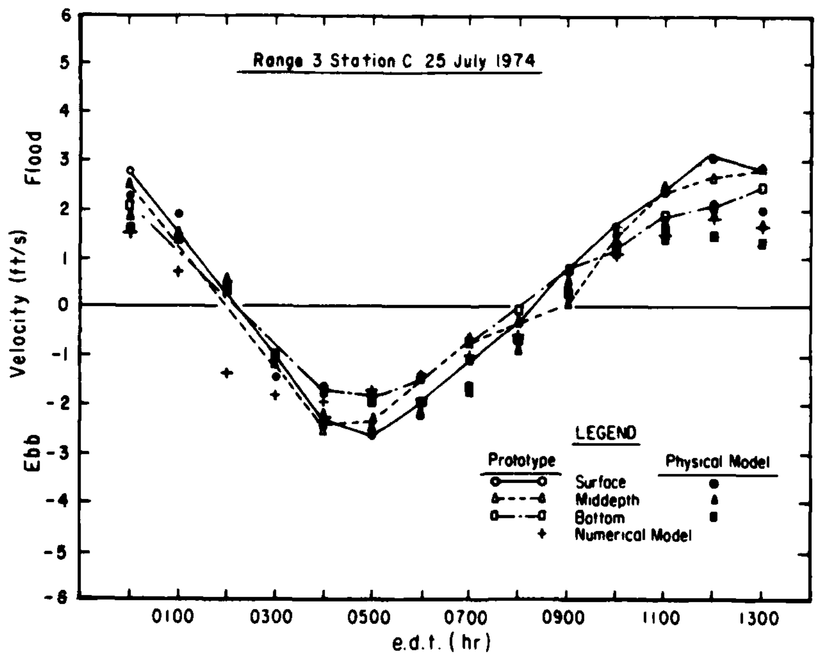


Figure A-14. Observed and predicted velocities at range 3, station C.

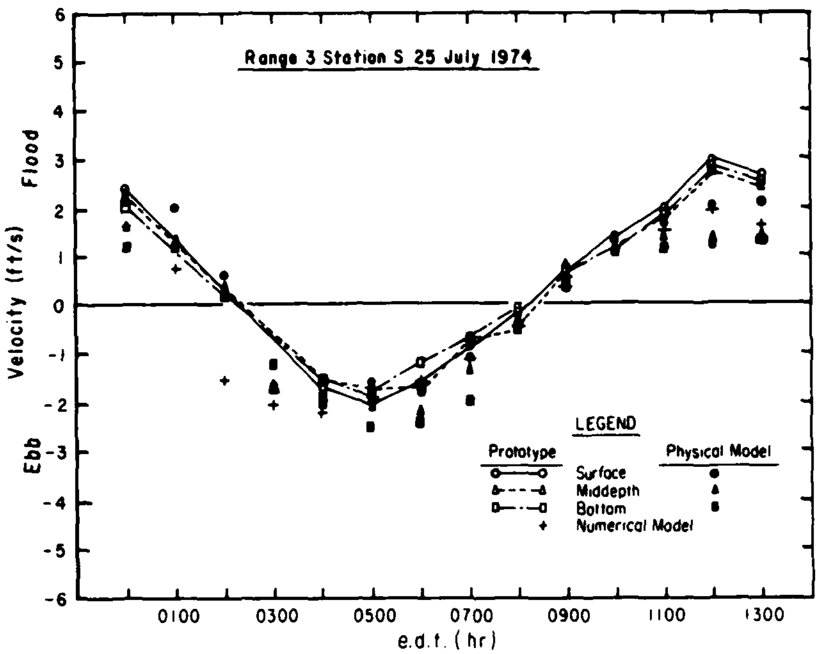


Figure A-15. Observed and predicted velocities at range 3, station S.

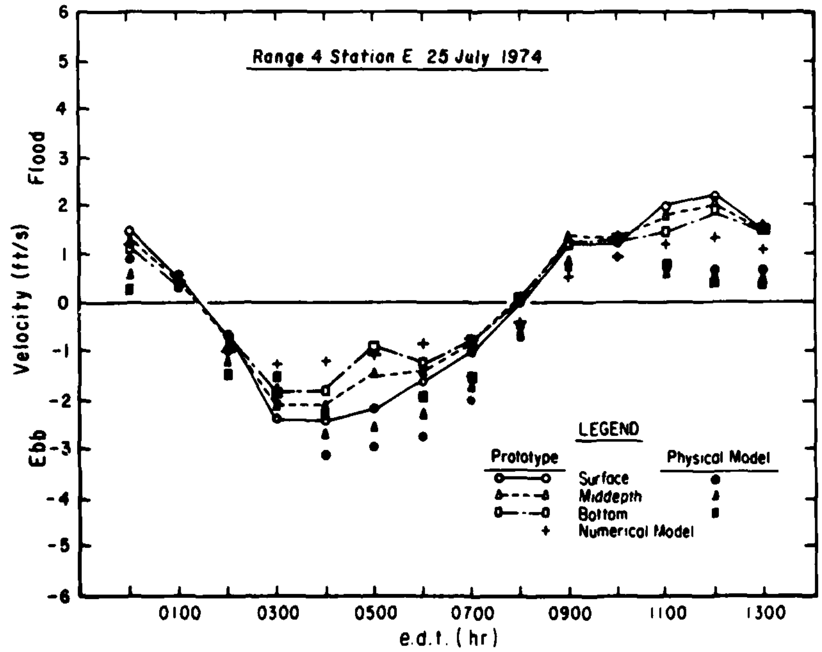


Figure A-16. Observed and predicted velocities at range 4, station E.

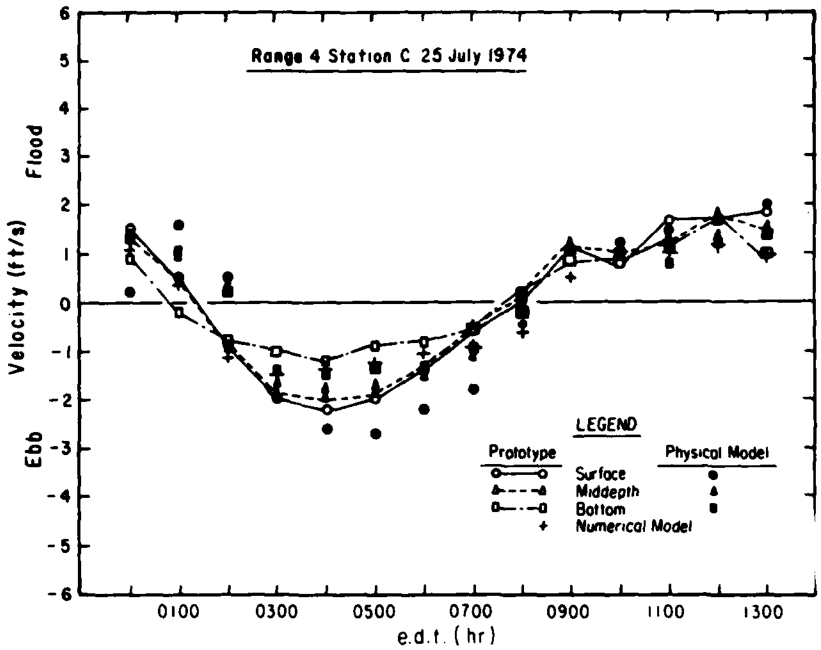


Figure A-17. Observed and predicted velocities at range 4, station C.

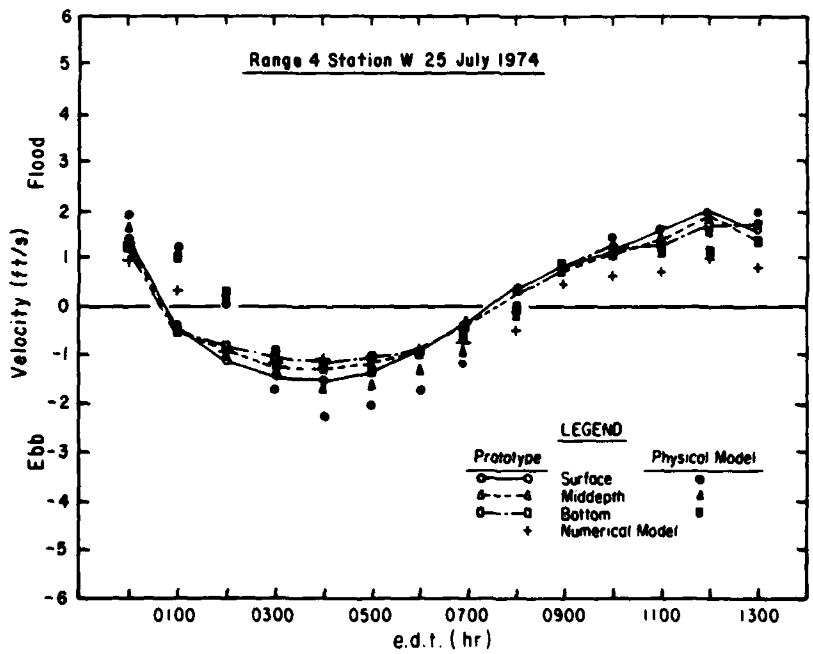


Figure A-18. Observed and predicted velocities at range 4, station W.

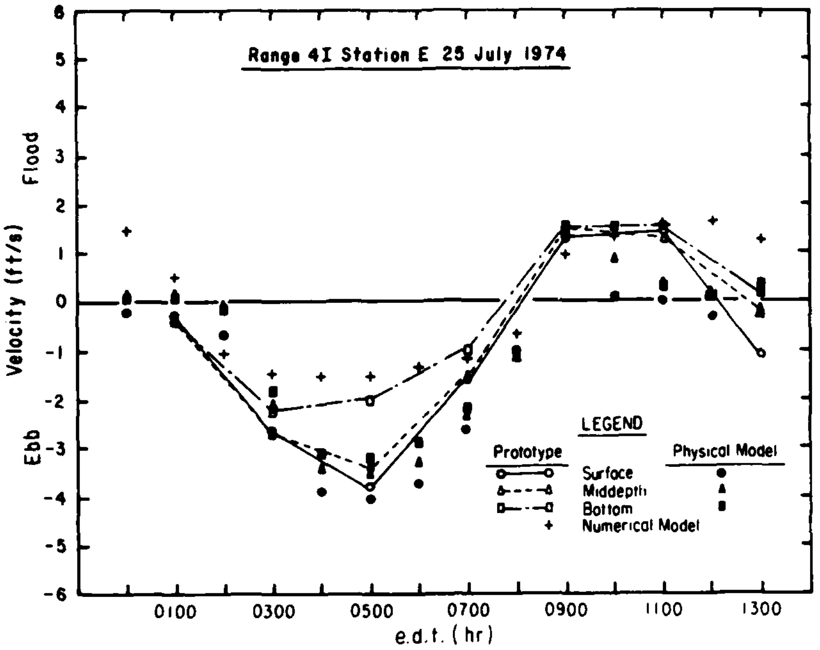


Figure A-19. Observed and predicted velocities at range 4I, station E.

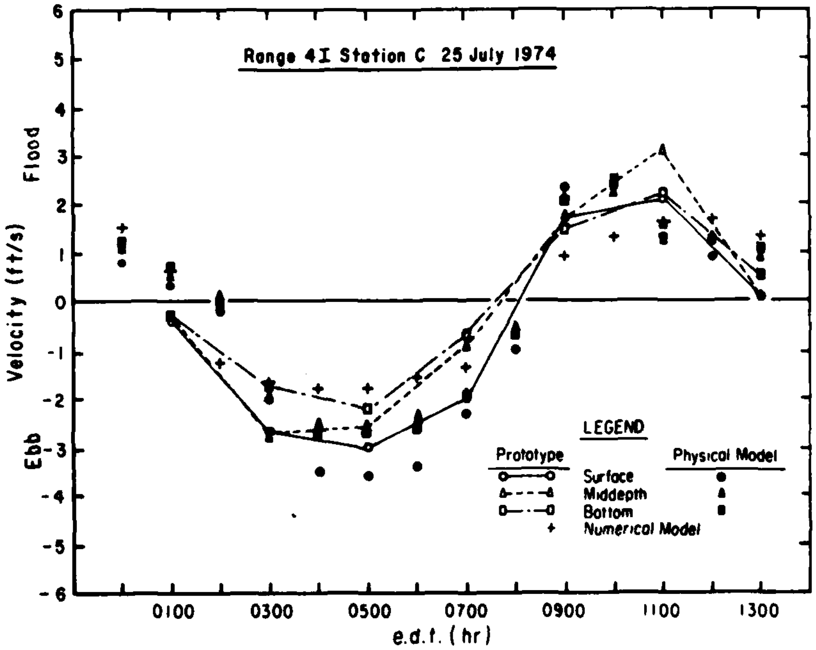


Figure A-20. Observed and predicted velocities at range 4I, station C.

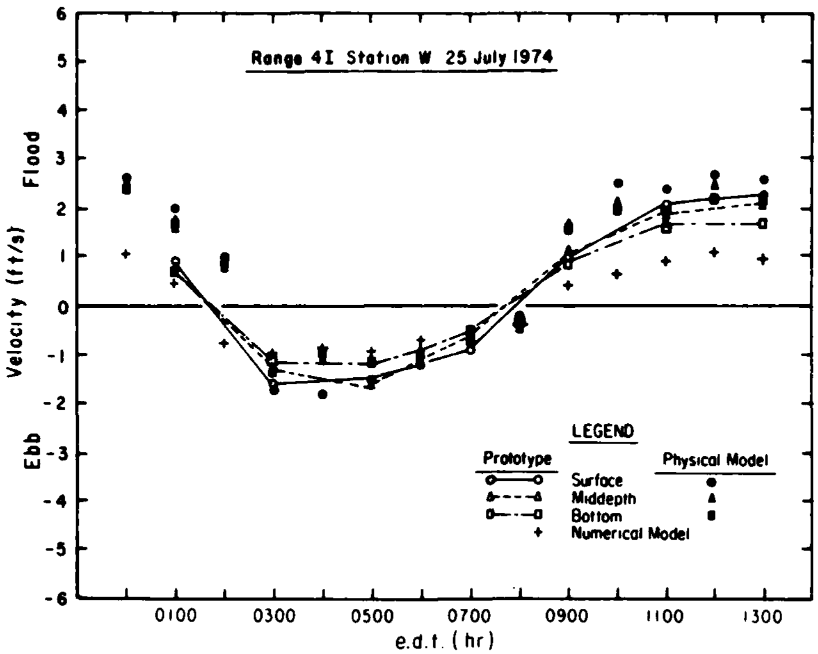


Figure A-21. Observed and predicted velocities at range 4I, station W.

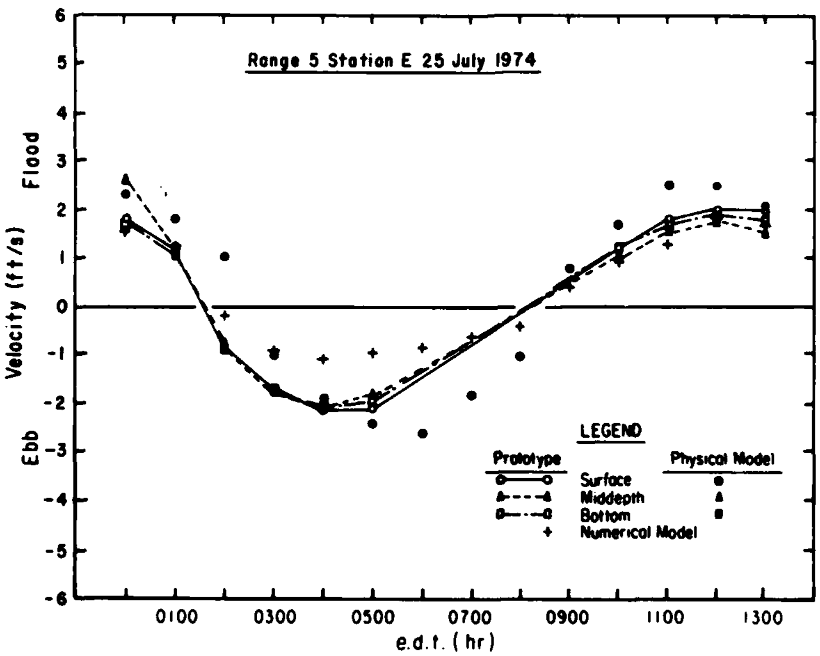


Figure A-22. Observed and predicted velocities at range 5, station E.

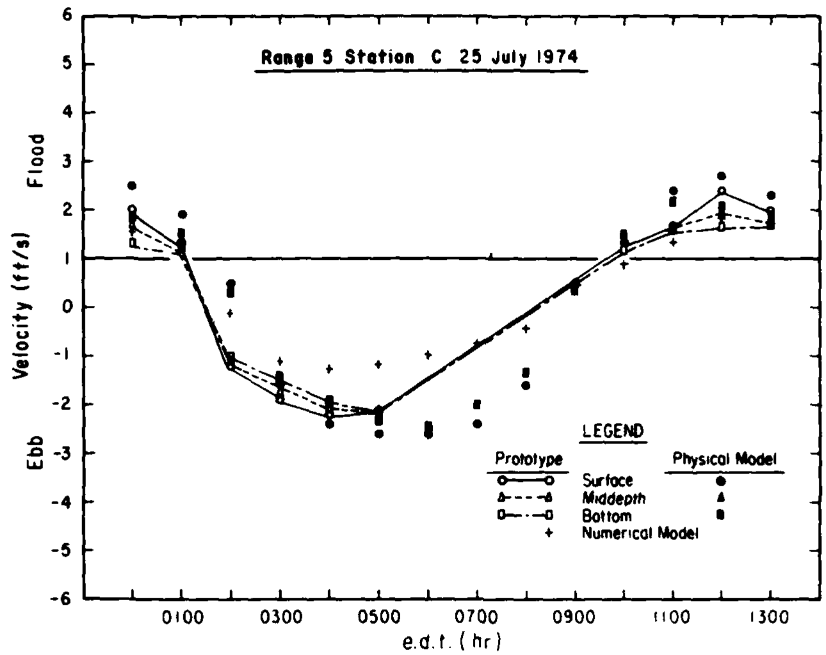


Figure A-23. Observed and predicted velocities at range 5, station C.

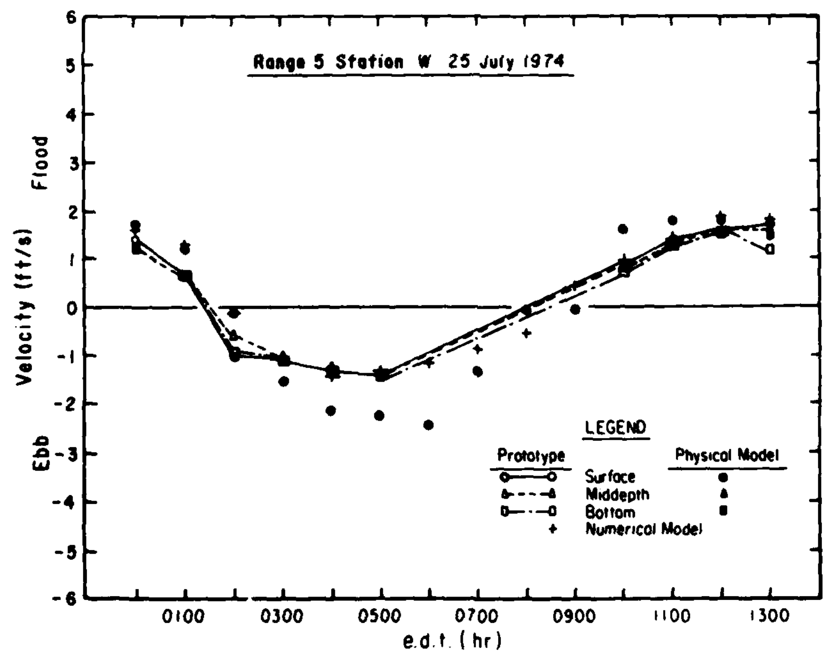


Figure A-24. Observed and predicted velocities at range 5, station W.

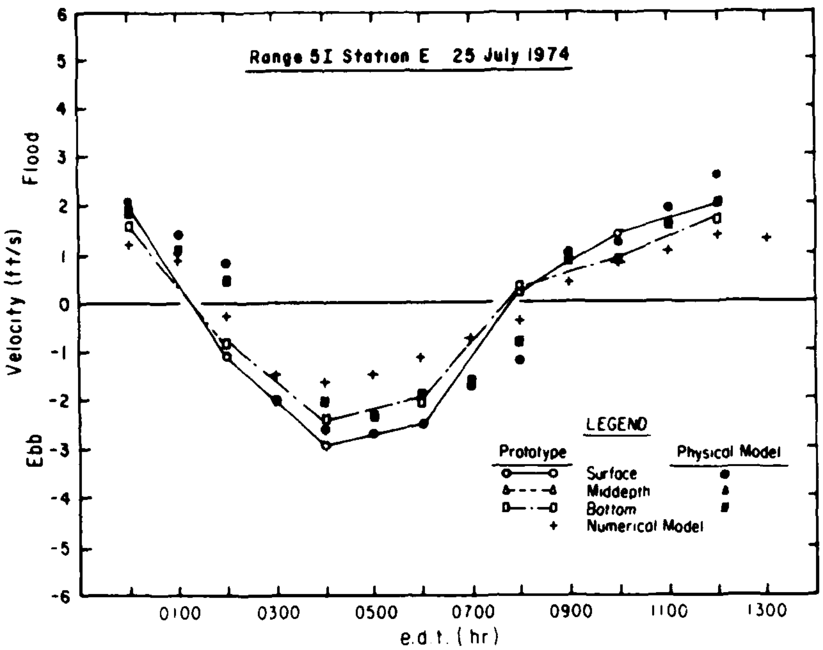


Figure A-25. Observed and predicted velocities at range 5I, station E.

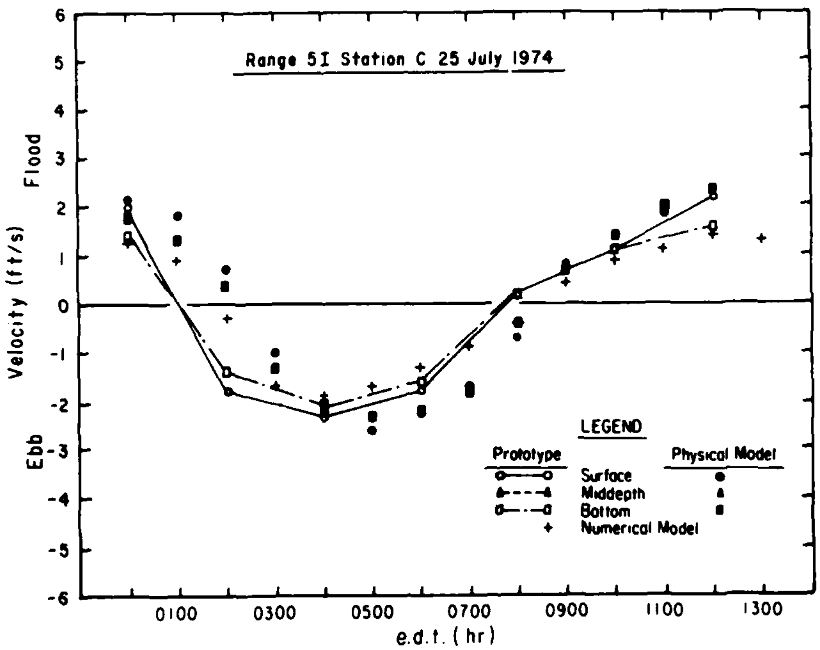


Figure A-26. Observed and predicted velocities at range 5I, station C.

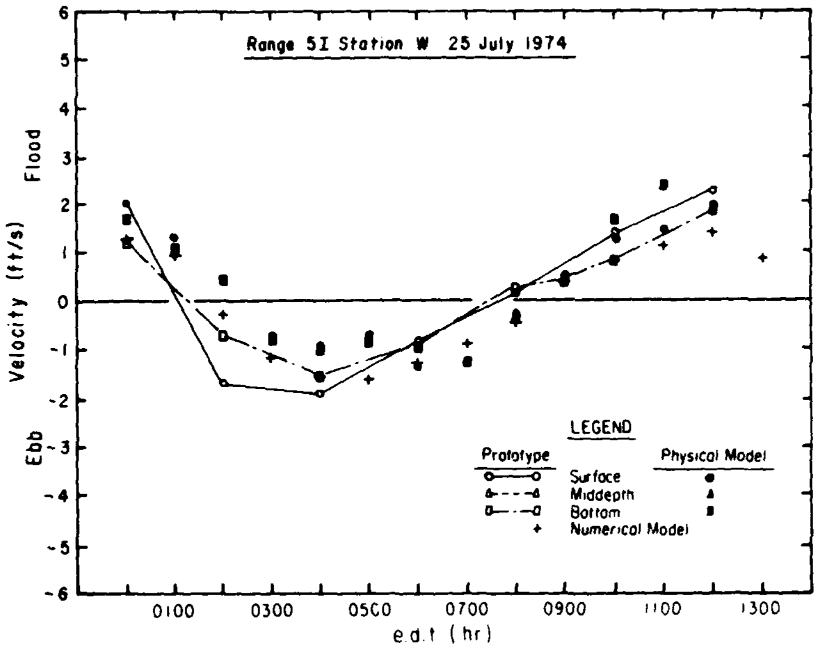


Figure A-27. Observed and predicted velocities at range 5I, station W.

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Bibliography: p. 35.
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